Vol. IX. — No. 5. May 1927.

BULLETIN
OF THE
INTERNATIONAL RAILWAY CONGRESS
ASSOCIATION
(ENGLISH EDITION)

[ 624. (01 ]

# Notes on the impact effect on metal bridges,

By R. DESPRETS,
ENGINEER AT THE BELGIAN NATIONAL RAILWAY COMPANY,
PROFESSOR AT THE UNIVERSITY OF BRUSSELS.

Figs. 1 to 14, pp. 370 to 391.

## I. — Introduction

The following notes, which are not intended to be a new contribution to the complex and difficult subject of impact effects on metal bridges, give concisely a general summary of the work done and the experiments carried out in this little known field. The idea of writing them was suggested by a desire to bring to notice the investigation into this subject made by the Government of India, with a view to drawing up Rules for the calculation of railway bridges which should be based upon the most recent technical knowledge.

The final report, full extracts from which will be found below, was published by the Government of India under the title : *First and Second Interim Reports of the Bridge Sub-Committee, 1925, on « Impact » and « Revision of the Bridge Rules » with connected papers — and — Proposed Bridge Rules for 5 ft. 6 in. gauge railways*, Calcutta. « Government of India Central Publication Branch », 1926.

This report is one of the latest and most informative on the subject of impact, and especially so from the practical point of view. It amplifies in a useful way the very important American prewar report and the English report of 1921. The most valuable conclusions of these two reports are also given below to shew what had been done previously.

These reports agree in attributing to periodic impulses the impact or dynamic increase of stress in railway bridges. These impulses are provoked by the centrifugal force resulting from the excess counterbalance required to balance in the horizontal plane, the inertia forces due to the reciprocating parts — piston, piston rod, crosshead, and part of the connecting rod. The effect of this centrifugal force on any particular bridge depends upon the speed of the locomotive. It increases to the *critical speed* at which the period of rotation of the

wheels counter-balanced corresponds with the period of vibration of the bridge itself. This speed is the only one at which the vibrations synchronise. In certain cases this synchronism would result in an increase in amplitude of the vibrations, possibly to a dangerous extent. Above or below this critical speed, the vibrations do not synchronise, and the impact effect diminishes.

The reports mentioned above agree in ascribing the effect chiefly to the periodic blows due to the excess counter-balance and, to a smaller extent, to the shocks due to the rail joints, to wheel flats, and to badly laid track.

Generally speaking, for both moderate and long span bridges it is understandable that the effect of the joints would be small in the case of properly maintained track.

The reports do not indicate that the design of the structure has any marked influence. It is, however, probable, that the type of construction does affect the results, and that if the track is laid on ballast the vibrations are more likely to be damped out than if the rails are laid on longitudinal stringers and cross girders.

The German regulation of 1926 takes these differences into account. The form of structure has more influence on the effects of shocks due to the joints or to wheel flats, than on those caused by the periodic centrifugal force of the counter-balance. A characteristic difference in the *methods of making the measurements* is that the impact is determined by measuring the deformation as a whole (total deflection of the structure) in the American and Indian reports, whereas the English report is based on the measurement of local strains. The Indian Reporters point out with reason that the deformation of the whole structure gives a better average of the phenomena, localised measurements being vitiated by local effects, secondary stresses, vibrations in the instruments themselves, etc. The American and Indian reports shew that the impact effect, calculated in accordance with the two methods, follows the same law, and that the impact determined by local measurements exceeds the former by only about 10 %. A new point brought out by the Indian report is the existence of another maximum impact effect at a speed corresponding to a period of rotation double that of the period of vibration of the bridge. The maximum corresponding amplitude is, however, less than at the true *critical speed.*

The three reports, all of Anglo-Saxon origin, now form the most valuable and detailed documents we have on the question of impact on metal bridges.

It would, however, be a great mistake to think that the effect of the speed of the loads on metal bridges has only been considered recently. In 1847, Queen Victoria's Government appointed a Commission to investigate the question. The Commission reported in 1849 : *Report of the Commissioners appointed to inquire into the application of iron to railway structures, presented to both Houses of Parliament by command of Her Majesty, London, 1849,* certain of the conclusions being confirmed many years later by theoretical deductions made by French engineers. The new feature in the recent experimental investigation is the new orientation of ideas, the impact effects being attributed to causes formerly neglected, causes once considered important now being put aside. In early days little or no importance was attached to the periodic effect of the counter-balance, and the theoretical investigations all dealt with the effect of speed itself. The speed

effect can only be seen in the inertia forces due to the vibration of the structure and in the unimportant centrifugal forces caused by the deformation of the structure. A great English mathematician, Stokes, a Professor at Pembroke College, Cambridge, and the French Engineers, Philipps, Renaudot and Bresse, wrote valuable papers on the subject. A summary of a paper by Renaudot is given below, and the remarkable agreement between certain of his conclusions and those in the *Report of the Commissioners*, and the idea of treating the supplementary effect of the speed as a force acting vertically on the bridge girder are pointed out (Bresse).

The effect of speed can also result in parasitic movements of the locomotive on its springs, causing periodic increases or reductions of the load. These movements themselves and especially those of nosing and rolling, vary indirectly with the yielding of the joints. If the line is maintained in proper order the influence of these parasitic movements will be small. Practice and theory agree in shewing that the *influence of speed under these aspects is small* and, in fact, the tests definitely prove that the regular periodic shocks are those of prime importance.

Mr. Deslandres, in a paper in the *Annales des ponts et chaussées*, 1892, called attention to these regular vibrations and attributed them principally, so far as railway bridges were concerned, to shocks at the rail joints. It is rather remarkable that in early days no great importance was attributed to the periodic vibrations due to the counter-balance, seeing that at that time most of the engines had two cylinders, and consequently caused the greatest impact. At the present time, on the other hand, the question tends to become of less importance owing to the increasing number of completely balanced locomotives whether compound or electric.

The notes to be given are therefore a review of the papers on impact, and of the most interesting conclusions thereon. It is well to call once again to notice the forgotten work of Philipps, Renaudot and Bresse, whose theoretical investigations were important, and some of whose conclusions are still of value to-day.

Pioneers are too frequently forgotten : new ideas are rare and too often, without knowing it, the work done by early investigators is repeated.

There has been no lack of mathematical investigations into the vibration of girders over which a uniformly distributed load, or a single moving load passes at speed. Whilst they cannot be said to have served no useful purpose, there is no doubt that they were of little practical use. The real problem which takes into account the many periodic forces which cause vibration has not been solved : mathematically considered it would be extremely complicated. In this connection the work of Bresse, Resal, Lebert, Zimmermann, Timoschenko, Kriloff, etc., should be noted. The Pencoyd is the formula most used in America for many years for the calculation of metal bridges :

$$I = \frac{300}{300 + L}.$$

For

$$L = 0, \quad I = 100\ \%$$
$$L = 100 \text{ feet}, \quad I = 75\ \%.$$

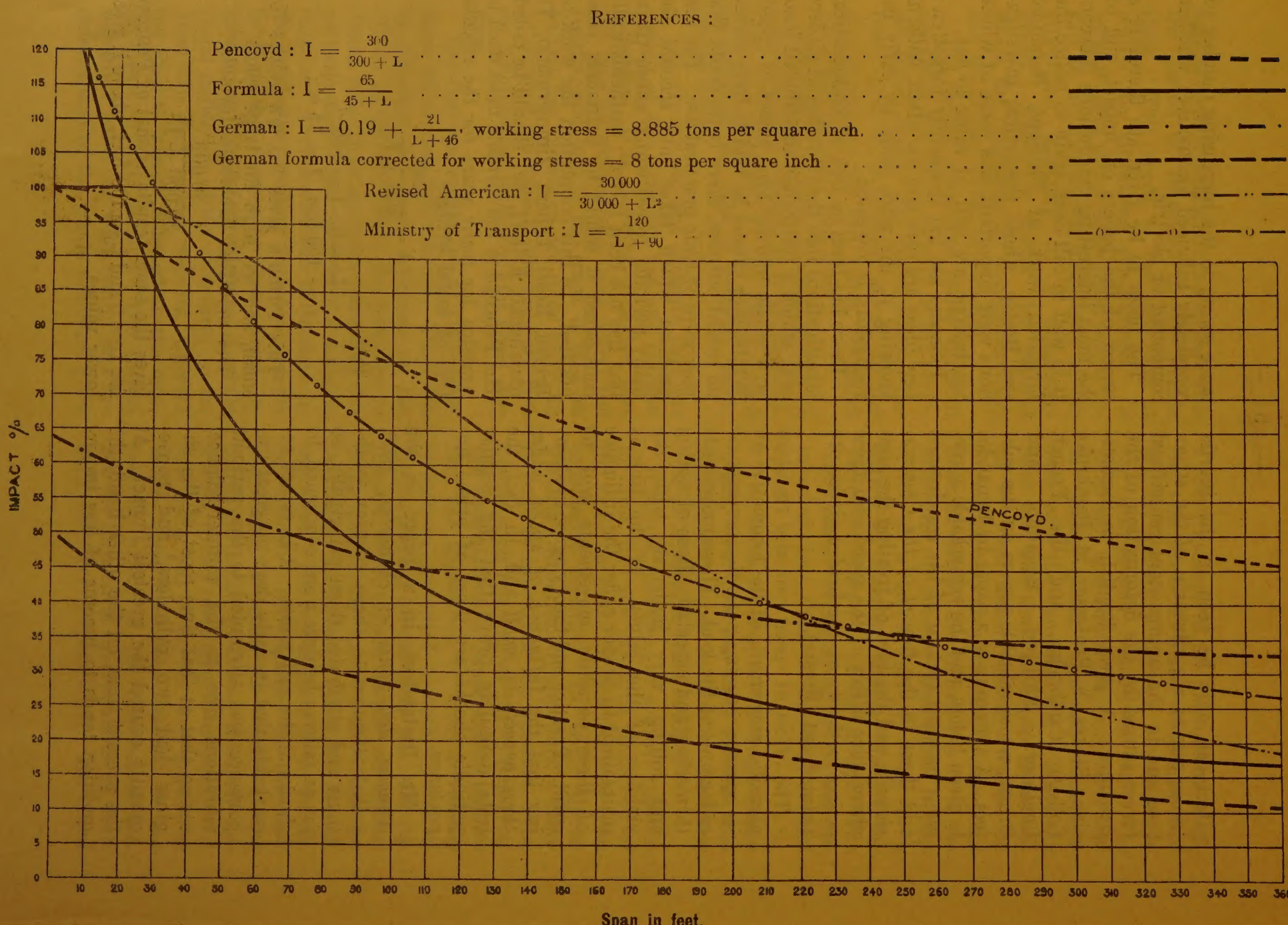
This formula gives too low values for small, and too high for large spans.

The modified American formula

$$I = \frac{30\,000}{30\,000 + L^2}$$

gives the same results as that of Pencoyd for L = 0 and L = 100 feet; above it gives lower values.

Fig. 1. — **Comparison of impact curves**



The English formula $I = \frac{120}{L + 90}$ gives $\frac{4}{3} = 133\ ^\circ/_\circ$ for $L = 0$, and about $65\ ^\circ/_\circ$ for $L = 100$ feet.

In these formulæ L is in feet and I is the *increase* of effect of the moving loads.

In the German regulations of 1922 the co-efficients of impact (Stosszahl) are

$$I = 0.19 + \frac{21}{l + 46}$$

for metal bridges with sleepered track laid directly on the longitudinal stringers or on the main girders.

For

$l = 0$ m., $I = 65\ ^\circ/_\circ$;
$l = 30$ m. (100 feet), $I = 45\ ^\circ/_\circ$.

These values of I are much lower than those given above.

The German regulations also make a great difference between track laid on longitudinal stringers and that on ballast. In the latter case

$$I = 0.11 + \frac{56}{l + 144}.$$

For

$l = 0$, $I = 50\ ^\circ/_\circ$;
$l = 30$ m. (100 feet), $I = 42\ ^\circ/_\circ$.

These values are therefore still further reduced. Above spans of 50 m. (164 feet) the constants are the same for all kinds of structures and I has a value of 30 % for 150 m. (492 feet) span.

In this case Pencoyd would give 40 %, and the English formula . . . . . . 22 %.

Different values may be attributed to the different forms of construction for small or moderate spans, as the critical speeds may not be reached, in which event the auxiliary phenomena of shocks at the joints and wheel flats may have a relatively great importance when determining the impact constant.

In conclusion, the impact should not be exaggerated, particularly as with the improvement in the balancing of locomotives the effect due to the principal causes will be reduced. The remark of the *Director of Engineering* in India that the co-efficient of impact determined for badly balanced locomotives ought not to be applied to the heavy rolling loads with modern large well-balanced engines, should not be overlooked.

## II. — Notes on the work done by Renaudot, Bresse, Souleyre and Deslandres.

§ 1. — *Renaudot*, in his paper « Etude de l'influence des charges en mouvement sur la résistance des ponts métalliques à poutres droites » (The influence of moving loads on the strength of metal bridges with straight girders) (*Annales des ponts et chaussées*, 1861-I), dealt with the stresses in a girder supported at each end with a continuous and uniformly distributed load in motion.

Philipps published a solution of the same problem for a single moving load in the *Annales des Mines*, 1855.

If $l$ is the distance between the supports A and B, $p$ the dead load per lineal metre, P the rolling load per lineal metre, the part of the girder covered in time $t$ being AC, the fundamental equation for each of the parts AC and CB is

$$EI \frac{d^2y}{dx^2} = M, \quad \epsilon = EI$$

and

$$\epsilon \frac{d^4y}{dx^4} = \frac{d^2M}{dx^2}.$$

I being the moment of inertia, and M the bending moment.

*For the part under the rolling load,* the unit loads are :

1. $p + P$, dead load + rolling load ;

2. $m \frac{d^2y}{dt^2}$ inertia force of the girder with rolling load ;

3. $m \frac{v^2}{R} = \frac{P}{g} v^2 \frac{d^2y}{dx^2}$ centrifugal force due to rolling load.

The equations for the parts under rolling load and under dead load only are :

*Part under rolling load :*

$$\epsilon \frac{d^4y}{dx^4} = p + P - \frac{P}{g} v^2 \frac{d^2y}{dx^2} - \frac{p+P}{g} \frac{d^2y}{dt^2}.$$

*Part under dead load only :*

$$\epsilon \frac{d^4y}{dx_1^4} = p - \frac{p}{g} \frac{d^2y}{dt^2}.$$

The plus $x$'s are counted from A to B, the plus $x_1$ from B to A and the plus $y$'s below the centre line of the $x$'s.

By demonstrating that at the point of junction the quantities $y$, $\frac{dy}{dx}$, $\frac{d^2y}{dx^2}$, $\frac{d^3y}{dx^3}$ were the same for the two sections, Renaudot ascertained the constants by integration.

If M and $M_1$ are the moments in the sections under moving loads and under dead load only, the relation $\frac{dM}{dx} = 0$ and $\frac{dM_1}{dx_1} = 0$ from which the abcissæ of the points where the M's are maxima can be established.

*Maximum value of* $M_1$. — $M_1$ is maximum for $\frac{dM_1}{dx_1} = 0$ corresponding to

$$x_1 = \frac{l}{2} + \frac{P}{p} \frac{\overline{vt}^2}{2l}$$

for $t > 0$ $x_1 > \frac{l}{2}$, $x_1$ corresponding to $M_{1max}$ in the part *not under moving load in front of the head of the train.* — It will meet it at the moment that $l - x_1 = vt$, that is, when

$$x_1 = \frac{p}{P}\left[\sqrt{1 + \frac{P}{p}} - 1\right] l.$$

For

$$p = P, \quad vt = 0.41\ l.$$

The point of maximum moment then moves back, follows the head of the train and attains its limit for $x = \frac{l}{2}$ at the centre of the girder.

If the point of the *maximum maxima moment* is desired for

$$\frac{dM}{dx} = 0 \text{ and } \frac{dM}{dt} = 0$$

we find

$$x = \frac{l}{2}\left[1 + \frac{Pv^2}{8Mg}\left(\frac{l}{2}\right)^2\right].$$

This value of $x$ corresponds also to $vt > l$, the head of the train having *passed* the end of the girder

$$vt = l\left[1 + \frac{1}{2.\ 3.\ 4.} \frac{(P+p)\ v^3 l}{\epsilon g}\right].$$

*The maximum moment of the maxima therefore occurs beyond the centre of the girder.*

This fact was already brought out in the *Report of Commissioners, London, 1849.*

The Reporter stated it had been noticed that when the load was moving, the points of greatest flexure, and still more so the greatest stresses, did not remain at the centre of the bar but moved towards its end.

When the bars were broken by a moving weight, the fracture always occurred beyond the centre. *As a first approxi-*

*mation*, the maximum moment occurs when

$$vt = l,\ x = \frac{l}{2}.$$

$$M_{max} = -\frac{1}{8}(p + P)\,l^2\left[1 + \frac{1}{6}\frac{Pv^2}{\epsilon}\frac{l^2}{g}\right].$$

The ratio of the dynamic moment to the static moment is

$$1 + \frac{1}{6}\frac{Pv^2}{\epsilon}\frac{l^2}{g}.$$

When the load is concentrated at a point (problem dealt with by Philipps) the supplementary factor would be doubled and have the co-efficient of 1/3 instead of 1/6. When considering the *deflections*, as in the case of the *moments*, it will be found that

$$\frac{Yd}{Ys} = 1 + \frac{1}{6}\frac{Pv^2 l^2}{\epsilon g}.$$

If this ratio is put into figures it will be seen that *the influence of the speed on the increase in the static deflections is generally insignificant and usually inappreciable.*

The effect upon the ratio of the dynamic and static moment is also small.

* * *

§ 2. — *Bresse*, in his *Cours de mécanique appliquée* (*Lectures on applied mechanics*) *given at the « Ecole des Ponts et Chaussées »*, Paris, 1880, considered the equilibrium diagram for a girder under a moving train of indefinite length. He presupposed the girder to have a live load without speed, that is to say, that the rolling load was gradually getting into motion. The girder is stressed by the dead load and by the moving load included $p$, and by the centrifugal force due to the moving load.

The differential equation of equilibrium is

$$\epsilon\frac{d^4y}{dx^4} + q\frac{v^2}{g}\frac{d^2y}{dx^2} - p = 0$$

$q$ is the fraction of the moving load included in $p$ : On integrating, the moment

$$M = \epsilon\frac{d^2y}{dx^2} = \frac{1}{2}p(x^2 - a^2) - q\frac{v^2}{g}y.$$

The bending moment is made up of the ordinary moment under vertically acting loads and of the moment resulting when presupposing the *part loaded vertically by two forces acting at its ends* $= q\dfrac{v^2}{g}$.

Having considered certain conditions, Bresse determined the critical speed by the equation

$$q\frac{v^2}{g} = \frac{\epsilon\pi^2}{16a^2}$$

where $a$ is half the span of the girder.

§ 3. — This result can also be arrived at by treating the girder as a part stressed by the vertical supplementary force $\dfrac{qv^2}{g}$ and by an axial force causing bending moments equal to those of the permanent load and of the vertically acting moving load.

This axial force referred to the centre of the girder would be X such that

$$Xf = \frac{pl^2}{8},\ f = \frac{5}{24}\frac{pa^4}{\epsilon},\ X = 2.4\frac{\epsilon}{a^2}.$$

So that the girder shall not buckle under the action of these two forces it is necessary that

$$\frac{pa^2}{2f} + \frac{qv^2}{g} < \frac{\epsilon\pi^2}{4a^2}\text{(Euler's loading)}$$

where

$$\frac{qv^2}{g} < \frac{\epsilon}{a^2}\left(\frac{\pi^2}{4} - 2.4\right)\text{ that is} < \frac{\pi^2}{16}\frac{\epsilon}{a^2}\text{(Bresse).}$$

The maximum working stress being R,

$$R = \frac{M}{\frac{I}{v}} = \frac{MH}{2I} = \frac{pa^2}{4}\frac{H}{I}, \frac{I}{a^2} = \frac{pH}{4R}$$

$$\frac{qv^2}{g} = \frac{\pi^2}{16} E \frac{I}{a^2};\ v^2 = \frac{\pi^2}{16}\frac{p}{q} g \frac{E}{4R} H$$

$v_1$ the critical speed is proportional to

$$\sqrt{H}, \text{ to } \sqrt{\frac{p}{q}} \text{ to } \sqrt{\frac{1}{R}}.$$

The calculations shew that this speed is much higher than those attained in practice.

When analysing the additional dynamic moment of Renaudot by considering the equivalent vertical load, it is found that this additional moment is equal to

$$\frac{Pv^2}{g} \times \frac{1}{48}\frac{(p+P)l^4}{\epsilon},$$

the equivalent upright load is equal to $\frac{8}{5}$ of $\frac{qv^2}{g}$, the load given by Bresse.

In effect

$$\frac{1}{48}(p+P)\frac{l^4}{\epsilon} = \frac{8}{5} \times f.$$

It is as though the load as given by Bresse acted with a leverage at the centre equal to $\frac{8}{5}$ of the ordinary deflection.

This result can be understood by noting that Bresse supposed that the load came into action *slowly* without any vibration due to the inertia of the girder.

* * *

Another note should be given on the conclusions postulated by Renaudot.

When the head of the train has reached a point near the centre (0.41 $l$ for $p = q$) the point of maximum stress moves towards the centre but behind the head of the train.

In a recent note (1) on the tests of railway bridges in Russia we pointed out that the deflection is out of phase with the actual position of the load. Renaudot's mathematical conclusion, made over 60 years ago, forecast this result.

* * *

§ 4. — Mr. Souleyre, Bridge and Roads Engineer, in his paper « Action dynamique des charges roulantes sur les poutres rigides qui ne travaillent qu'à la flexion » (Dynamic action of Moving Loads on rigid girders working under bending only), (*Annales des ponts et chaussées*, 1889-2) further examined the dynamic effects taking into account speed alone. He considered that « irregularities in the surface moved over (projections in stone and wood roadways, imperfect joints in railway lines) caused the load to drop suddenly, and thereby gave rise to shocks without, however, attributing to them and above all those occurring regularly, any real and substantial importance. His paper is a mathematical study of the problem already dealt with. He made the valuable remark that all previous theories *supposed that the dynamic effect increased indefinitely with the speed* whereas it is evident — *a priori* — that a moving load travelling across a girder at infinite speed would produce no appreciable effect, and that there is consequently a speed which corresponds to the maximum effect for a given load on a given girder.

In his conclusions the confirmation is found of a previous conclusion, that *the maximum dynamic effect always occurs*

(1) See *Bulletin of the Railway Congress Association*, December 1926, p. 1074.

*beyond the centre of the girder in the direction of movement of the rolling load.*

* * *

§ 5. — Mr. Deslandres in an article published in 1892-2 in the *Annales des ponts et chaussées* called attention to the particular importance of *regularly occurring shocks in metal bridges.* « Tests have shewn beyond question the importance of the periodicity of *relatively light blows* to which a bridge may be subjected : but this periodicity has no effect unless the *period of vibration of the bridge is the same as the interval between two successive shocks.* »

In the case of railway bridges, he attached most importance to the periodic shocks at the joints. If $n$ is the speed in kilometres an hour, the speed per second is $\frac{n}{3.6}$. If the axles of goods wagons are $d$ metres apart, the interval between two successive shocks is $\frac{d \times 3.6}{n}$. If this interval equals the period of vibration of the bridge itself, the bridge will be subjected to periodic shocks and the amplitude of the vibration will be able to increase. The critical speed $n$ resulting therefrom would be equal to $n = \frac{3.6d}{T}$.

For example, if $d = 3$ m., $T = 0^s27$, $n = 40$ km. per hour.

### III — Impact tests on railway bridges [1].

Below are given extracts from the American report :

*Principal causes of impact.* — The principal causes are :

1. Excess counterbalance in the wheels;
2. Track with defective joints, or in bad order;
3. Flats or defects on tyres;
4. Wheels out of round;
5. Speed of the rolling load;
6. Effect of supplementary deformations on the magnitude of the applied loads.

*Impact* includes the *total* increase in stress due to the application of loads at *speeds.* When this dynamic increase is due to bad joints or wheel flats, the effect is that of a blow on the structure. So far as the horizontal balancing of the inertia forces of the reciprocating parts, by additional counterbalance weights, is concerned, the effect is that of successive periodic impulses.

The tests shew that for a line in good order with material in good condition, the principal cause of the impact effect is the excess counterbalance of reciprocating parts (two-cylinder locomotives). The resulting centrifugal force is considerable at high speeds. At the speed of 80 miles per hour, this force may be as much as the static load of the wheel. The knowledge of these effects should affect the construction of locomotives by leading to alteration in the methods used for counterbalancing.

The succeeding periodic impulses due to the supplementary centrifugal forces give rise to additional vertical deformations, and if the period of rotation of the wheels coincides with the period of vibration of the structure itself, these deflections may occur at the same time and become cumulative. The corresponding speed of movement of the rolling load is the *critical speed.*

This critical speed in practice can only be attained on bridges of over a certain span.

(1) *American Railway Engineering and Maintenance of Way Association Bulletin*, No. 125, July 1910.

*Period of vibration of the structure.* — This period can be expressed by the formula

$$T = \sqrt{\frac{w + p}{p}} \times d$$

$T$ = period in seconds,
$w$ = dead weight per foot in tons,
$p$ = rolling load per foot in tons,
$d$ = deflection in inches under the rolling load.

*Critical speed.* — During the passage of a train, the total weight of the bridge including the rolling load varies : *the frequency of the vibration* also varies. It follows that synchronism between the movement of the locomotive and the vibration of the bridge can only occur during a relatively short time, and that consequently the vibration can only be cumulative over a very short time, four or five periods at most.

The critical speed is specially important in the case of bridges of spans exceeding 100 feet. The diagrams *shew clearly that for such spans the maximum impact is due to the cumulative vibrations in the neighbourhood of the critical speed* (1).

*Theoretical calculation of impact.* — Supposing for extreme simplicity, the case of a girder with its weight concentrated at its centre with the periodic centrifugal force acting at the same point, the supplementary deflection may be expressed by

$$x = 2\pi n \frac{Gr}{2W}$$

where

G = weight of the counterbalance;
W = weight of the girder;
$r$ = radius of rotation of the counterbalance;
$n$ = number of revolutions after the movement began.

(1) Diagrams in the American report.

No account is taken of any damping out of the vibrations by passive resistances, absorption of energy, etc.

In practice the hypothetical synchronism on which this formula is based clearly cannot occur. Contrary to the result given by the formula, the vibrations *do not* build up indefinitely.

The *impact* is the ratio between the supplementary deflection $x$ and the static deflection $d$ due to the moving load :

$$I = 2\pi kn \frac{Gr}{2Wd} = k_1 n \frac{Gr}{Wd}$$

$k_1$ being a constant which takes into account the damping out of the vibrations.

The deflection $d$, when a *constant rate of work* is presumed, is more or less proportional to

$$l \times \frac{p}{w + p}$$

Furthermore, $n = \frac{l}{c}$, $l$ being the span of the bridge, and $c$ the circumference of a driving or coupled wheel.

I then is proportional to $k_1 \frac{Gr}{pcl}$.

The impact is therefore proportional to the moment of the counterbalance and inversely proportional to the moving load, to the span, and to the circumference of the driving or coupled wheels.

The expression $\frac{Gr}{c}$ could be called the co-efficient of the locomotive. The impact is proportional to this co-efficient.

The importance from the point of view of impact, of the supplementary weights used to counterbalance the inertia forces due to the reciprocating parts, has been clearly brought out by the tests with compound steam or electric locomotives. The impact with such locomotives is much smaller.

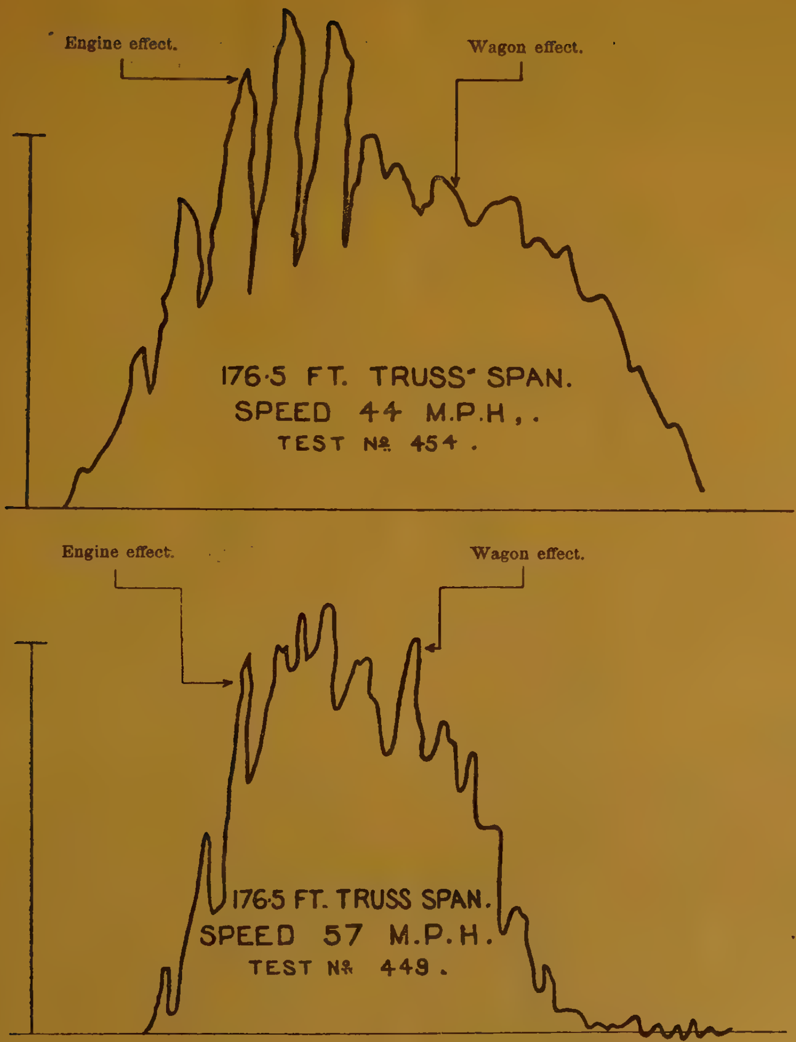



Fig. 2. — American Railway Engineering Association tests.

Typical deflectometer diagrams, shewing engine effect and effect of wagons.

*Note :* Diagrams 3 times the size of the original.

The tests were carried out in two ways :

1. By measuring the total deflection of the girders as a whole;

2. By measuring the stresses in certain members.

The results as a whole agreed, although the impact determined from the direct measurement of stresses in members is always higher than when obtained by measuring the deflection as a whole.

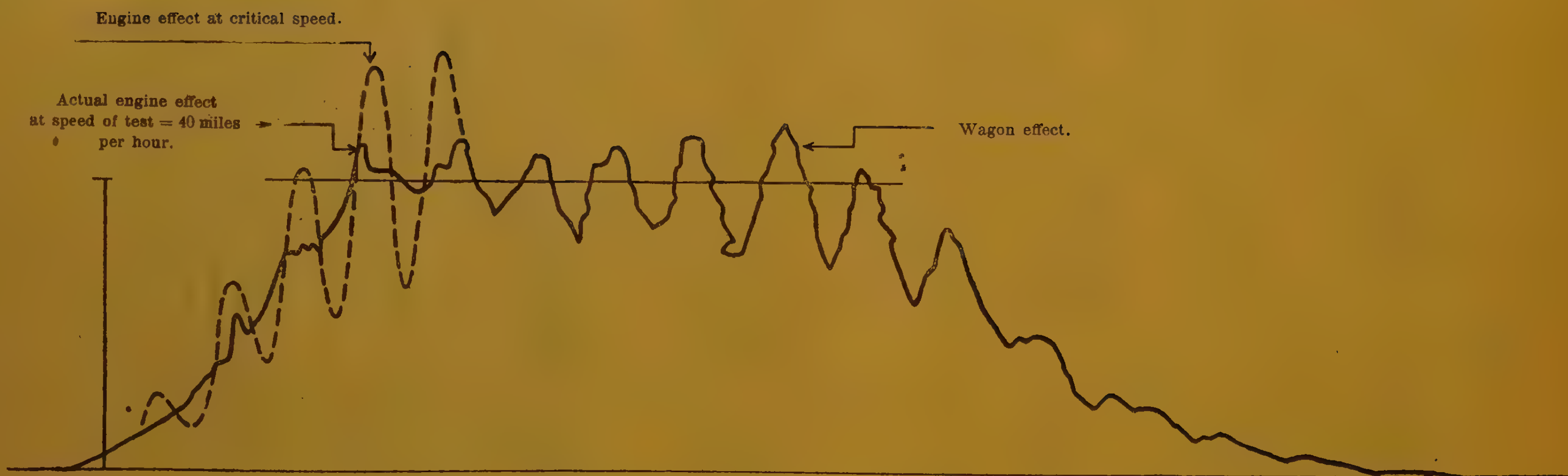



Fig. 5. — American Railway Engineering Association. — Test No. 1193.

126 0-foot truss span.

Example shewing a case where the wagon effect is greater than the engine effect at the actual test speed, but less than the engine effect at critical speed as shewn in dotted line.

This is explained firstly by the secondary stresses included in the direct measurement of the stresses in members, and secondly because the impact determined in this way is only the average of some parts, whereas when the deflection is measured as a whole, the result is the mean of the whole of the parts.

In short spans the maximum speed is usually lower than the critical, and the impact increases with the speed. For large spans the maximum impact occurs at the critical speed.

*Influence of the speed at which the loads are applied.* — When the static curve of the deflections is concave with regard to the base line, the movement of the load will cause additional centrifugal forces, and thereby supplementary stresses and deflections in the same direction.

When the bridge, before the moving load comes on to it, is cambered upwards to an extent corresponding to the statical deflection under the rolling load, the locomotives in motion will bring it back to the horizontal without inducing any additional centrifugal forces. The impact resulting therefrom will be nil in this case.

The exact determination of the actual effects of loads at speed is complicated when the other factors at work are taken into account. It would seem, however, that the effect of speed is small.

*General conclusions.*

1. When the track is well maintained, the principal cause of impact is due to the driving or coupled wheels being out of balance.

Small defects in the track appear to be unimportant for spans of more than 60 to 75 feet.

2. Supplementary cumulative deflections occur when the period of rotation of the wheels and the speed of vibration of the loaded bridge itself are the same. This speed is the *critical speed.* Above or below this speed the impact effect is smaller.

3. The critical speed falls as the span increases. The greatest impact occurs therefore at lower speed on long bridges than on short ones.

4. In the case of short span bridges when the train does not get up to the critical speed, the impact tends to remain constant so far as the counterbalance effects are concerned : the effect of defective joints and flats on wheels is, however, more pronounced.

5. The impact as given by direct measurements of the stresses is higher than when determined from the deflection as a whole. The two values follow the same general curve of variation.

6. For the same value of moving load the maximum impact is the same in the booms as in the diagonals. The same constants can be used with each.

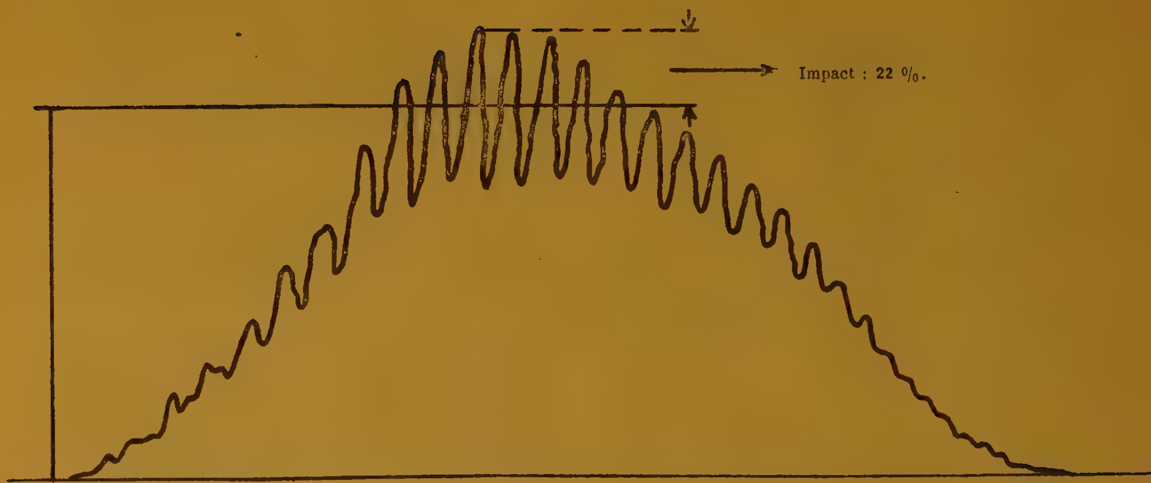
7. The maximum impact in the longitudinal girders is about the same as in the girders of equal span. For the floor girders and uprights the impact corresponds to that in a girder of a span equal to that of two panels of the main longitudinal girders.

The maximum impact may be expressed by the formula

$$I = \frac{100}{1 + \frac{l^2}{20\,000}}$$

where $l$ = span in feet.

9. The effect of different types of construction is chiefly felt in the flooring. An elastic structure, built up of long




300.0-foot truss span.

Speed : 25 miles per hour.

Test No. 890.

Calculated impact per cent : $I = \frac{1\,050 \times P_1}{(w+p) \times c \times d} = \frac{1\,050 \times 1.09}{2.96 \times 18.1 \times 0.97} = 22\ \%.$

Fig. 4. — American Railway Engineering Association. — Typical deflectometer diagram at critical speed.

*Note :* Diagram twice the size of the original.

cross members carried on longitudes widely separated, or one with ballast, gives lower constants than a rigid construction.

The flooring acts as an elastic cushion with regard to bad joints, wheel flats, etc. This effect is most noticeable in the longitudes, cross girders, uprights and girders of small span.

10. The effect of different types of construction is less evident in the parts of the main girders.

11. The impact due to speed itself is small when the line is well maintained.

12. The impact due to compound or electric locomotives is small and the vibrations are not cumulative.

13. The effect of wheel flats is most noticeable at high speeds on parts of the flooring.

## IV. — English report on impact, 1921.

**Advisory Committee for the revision of the Board of Trade requirements, 1914, in regard to the opening of railways.**

**Ministry of Transport.**

**Tests on railway bridges in respect of impact effect.**

Extracts from the English report of 1921 are given below.

12. *Secondary stresses.* — Some of the diagrams, principally for long span bridges, reveal vibrations at high frequency readily recognised and measured. These vibrations are noticeable especially

on the diagrams from the lower booms of the main girders but less so on those from the upper booms. Their frequency varies from 30 to 70 vibrations per second, which is much less than that of the recorder itself and higher than that of the girders. The frequency does not change when loads are applied. These vibrations clearly represent definite variations in stress due to rolling loads, and probably indicate secondary stresses.

The secondary stresses can be distinguished by carefully measuring the mean — maximum — of the high frequency vibrations.

When the results of the tests (excluding the secondary stresses) are drawn down, the curve is 40 to 50 % lower than the Pencoyd with which it generally agrees in form.

These tests might suggest that when measuring the increase in impact, any increase due to secondary stresses being excluded, *the maximum impact would not exceed 60 % of the moving load* for any span. Such a conclusion would be unjustified as the critical speed was not reached for all spans.

The tests seem to shew that secondary stresses which are frequently high cannot be avoided in the structures.

In long span bridges stresses of this kind must be looked for. We can admit for small spans when the maximum ordinates of the diagrams are used to determine the increases in impact that sufficient allowance is made to cover the secondary stresses. Large spans must be dealt with differently.

Normal secondary stresses will be covered by the impact, whereas abnormal stresses will not.

13. — An examination of the diagrams for small spans does not reveal any good reason for departing from the Pencoyd curve. It should be noted that speeds higher than the usual test speeds can be attained, and are not unusual with express trains. These speeds are lower than the critical for short spans, so that the impact constants may be higher in actual service.

No particular care appears to have been taken to get the driving wheels into the most unfavourable position, and consequently the maximum effects were not recorded.

The Pencoyd curves give too high figures for long spans if exceptional secondary stresses are not to be taken into account.

The Pencoyd figures, it must be noted, are too low for short spans.

It is interesting to note that with certain types of structures with ballasted tracks, the impact was found to be rather high. *It would therefore appear undesirable to reduce the constants in this case at the present time.*

14. — To sum up, the Pencoyd formula gives too small values for short spans and too high values for long spans.

The Commission suggests two new formulæ :

*a*) $$I = \frac{75}{L + 50}$$

were $I = 150$ when $L = 0$.

This formula gives a rather high value when $L = 0$ and does not agree with the results of the American tests when $L = 60$ feet.

*b*) $$I = \frac{120}{L + 90}$$

where $I = 133$ when $L = 0$.

This curve *best meets the requirements* giving values higher than those obtained from the Pencoyd formula for L of 0

to 50 feet, and 20 % less for L of 50 to 200 feet. This curve also covers the American tests mentioned above.

### V. — First interim report of the bridge sub-Committee 1925 on « Impact ».

*Calcutta, Government of India Central Publication Branch, 1926.*

A summary of the 1925 report of the Indian Railway Board already mentioned above on the determination of the impact effect is given below.

1. *Definition.* — The *impact* or the *dynamic increase* is defined by the increase in the effect of the *moving* load over the bridge at speed as compared with the effect of the same load at a crawl.

2. *Measurement.* — The changes can be measured by noting the additional *deflection* of the bridge considered *as a whole* or by the additional stresses in individual members.

The instruments used to measure the stresses did not record the average stresses, and furthermore the local stresses noted included all the secondary effects.

It has not been possible to separate the secondary effects in any measurements of stresses published up to now : consequently the figures obtained in this way cannot be used with full confidence.

On the other hand, the deflections of the structure as a whole are free from accidental variations of this kind, and the results obtained represent fairly well the averages for all the parts of the span.

For these reason the investigation has been confined practically to the determining of the *impact* allowance from measurements of the deflections of the whole structure.

3. — In particular cases direct measurements of stresses were made, the necessary precautions being taken to arrive at comparative mean values.

The conclusion come to is that the *impact* as determined by the measurement of the stresses in this way is 10 % higher than that obtained by measuring the deflections as a whole.

4. *Causes of the impact effect.* — The causes can be classified as follows :

1. Supplementary counterbalance weights have to be added to counterbalance in the horizontal plane the inertia forces due to the reciprocating parts, and cause at speed periodic centrifugal forces, the vertical component of which adds to or takes away from the load on the bridge. If this period which depends on the speed of the train, corresponds with the speed of vibration of the bridge itself under the rolling load, the periods synchronise and the effects add together to produce the maximum *impact effect.* The speed at which there is synchronism is the *critical speed.* The maximum centrifugal force is sometimes known in English as *Hammer Blow,* it being understood that it applies to the resultant centrifugal force for all the wheels. (This expression is not adequate seeing that the centrifugal force in question is periodic.)

2. The loads at speed move over a curve owing to the deflection of the bridge : this gives rise to centrifugal forces, the effects of which must be added to the original vertical forces. Supplementary deflections are caused in this way.

3. Defective track, bad joints and wheel flats also increase the *impact.*

5. — The effects of these different causes vary according as parts of the flooring, girders of short span or girders of long span are under consideration.

Of these causes the most important is that due to the horizontal counterbalancing of the reciprocating parts.

6. *Effects of the periodic forces.* — The counterbalance for the reciprocating parts is distributed amongst all or most of the driving or coupled wheels. On short span bridges only one of the coupled axles may be on the bridge, whereas on long span bridges all may be. In this latter case, the total effect of the counterbalancing as a whole must be taken into account, the resultant force being produced by a series of successive blows at a speed depending upon that of the locomotive.

The effect of the blow varies as the square of the speed, and for badly balanced engines can be as much as 20 tons, the blow occurring several times a second.

A span has a period of vibration of its own, and if this period is the same as that due to the counterbalance blows, the resultant effect becomes a maximum and may be serious.

The frequency of the vibration diminishes with the span : on short spans in practice the train cannot get up to the *critical speed* at which there is synchronism. Under such conditions the amplitude of the vibration cannot increase above a certain amount. The division between *long* and *short spans* comes from this consideration. Short spans are those on which trains cannot reach the critical speed at which the maximum impact occurs. The limit is somewhere between 50 and 75 feet depending upon the driving wheel diameter : as this is reduced so does the span lessen.

7. *Vibration of structures. Formula giving the frequency of vibration.* — An examination of the deflection diagrams given in the tests of the « Indian Railway Bridge Committee, 1921 », the « Great Indian Peninsular Railway in 1922 » and the « American Railway Engineering Association » shews that the dynamic increase in the deflection at the centre of the girder equals half the amplitude of the vibrations about the mean or static deflection line. Furthermore the maximum amplitude is reached when the train speed is equal to the frequency of vibration of the girder calculated from the formula

$$n_0 = \frac{1}{\sqrt{\frac{w+p}{p}\,d}} \quad . \; . \; . \; . \quad (1)$$

wherein

$n_0$ = frequency of vibration of the girder under moving load;
$w$ = dead weight per foot run;
$p$ = additional uniformly distributed load per foot;
$d$ = static deflection under moving load $p$.

8. — This formula for the frequency of vibrations under load has been proved to be correct within practical limits by the tests of the Indian Railway Bridge Committee.

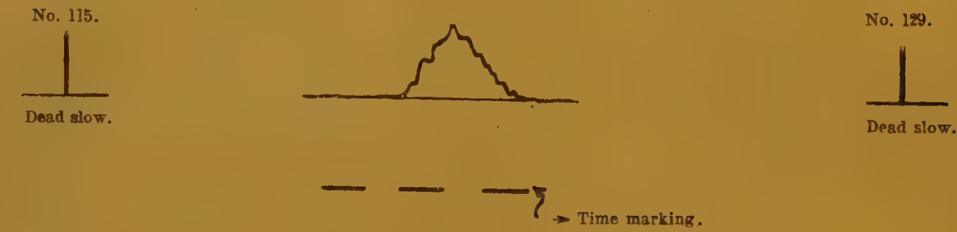
9. — Typical deflection diagrams are given below.

10. — A method of calculating the *Hammer blow* — maximum $P_1$ — the maximum centrifugal force corresponding to one wheel revolution per second will be found later on. This force would be $P_1 n^2$ at a speed corresponding to $n$ revolutions per second.

*Calculation of the dynamic increase of deflection giving the value of the impact.* — The data of the problems are :

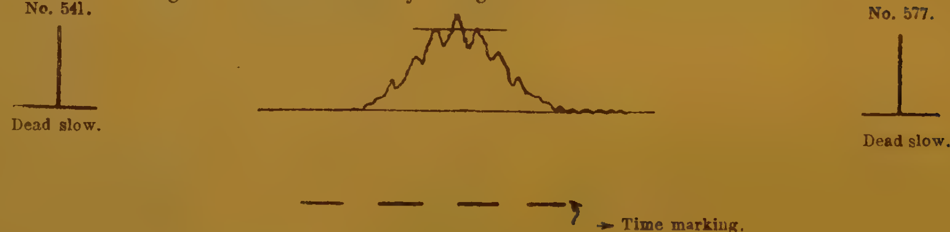
L = the span of the bridge;
$w$ = dead weight per foot run;
$p$ = equivalent rolling load per foot;
$P_1$ = maximum centrifugal force (*Hammer blow*) at one revolution per second;
$n_0$ = *critical frequence.*

Figs. 5 to 10. — Typical deflectometer diagrams at critical speed.



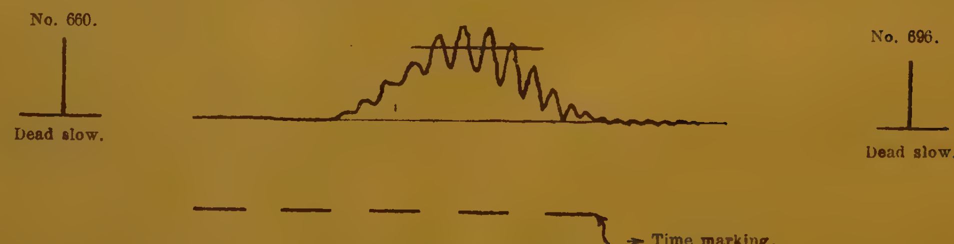

78.33-foot span.

Fig. 5. — Indian Railway Bridge Committee. — Test No. 127.




108.16-foot span.

Fig. 6. — Indian Railway Bridge Committee. — Test No. 565

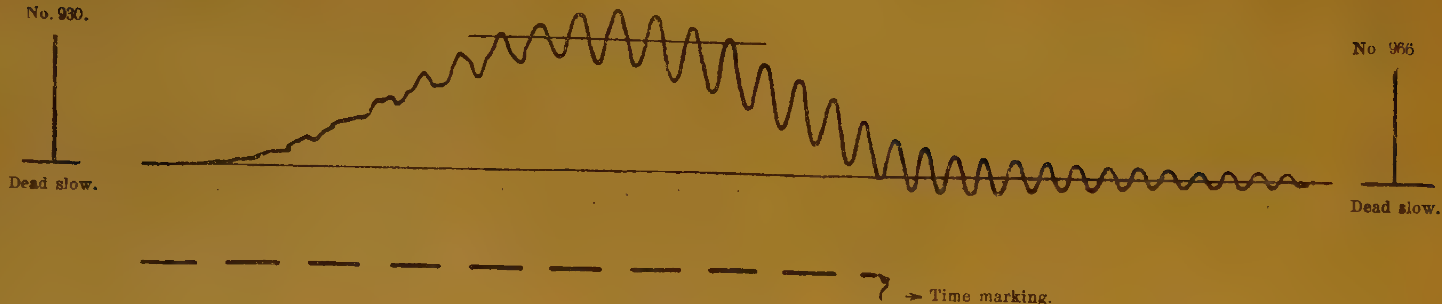



136-foot span.

Fig. 7. — Indian Railway Bridge Committee. — Test No. 680.

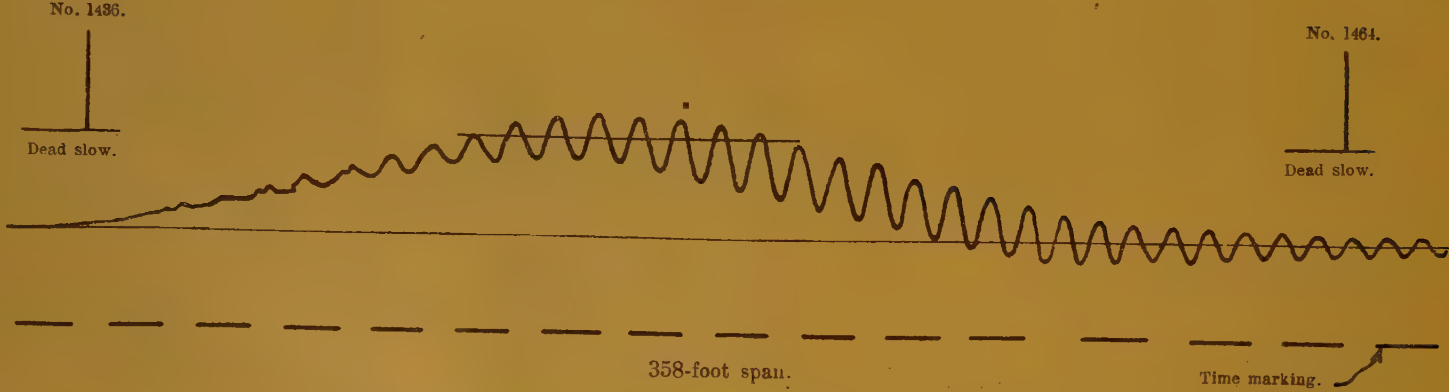

206-foot span.

Fig. 8 — Indian Railway Bridge Committee — Test No. 435.




257-foot span.

Fig 9. — Indian Railway Bridge Committee. — Test No. 938.




358-foot span.

Fig. 10. — Indian Railway Bridge Committee. — Test No. 1440

At the *critical speed* for each revolution of the coupled wheels two maximum impulses, one upwards and one downwards, each equal to $P_1 n_0^2$ (tons), have to be considered.

If the wheels are supposed to make N revolutions when crossing the bridge there will be 2N impulses, half acting upwards and half downwards. The effect of these forces increases as the wheel approaches the centre of the girder, when it attains the maximum and then decreases. As a rough approximation, the average effects may be taken as half the maximum. (The mean is actually greater than half, but a certain damping out of vibration has to be taken into account.) The effect of the 2N impulses in the two directions may be taken as equal to N impulses at the centre of the girder. An examination of the diagrams of the Indian Railway Bridge Committee and of the American tests shews that on the average the maximum deflection occurs when the train has traversed about two thirds of the span. The conclusion could be drawn from this that only $\frac{2}{3}$ N impulses need be considered as cumulative. The supplementary deflection due to these forces may be expressed by :

$$d_1 = \frac{P_1 n_0^2 L^3}{48EI} \times \frac{2N}{3} \quad . \; . \; . \quad (2)$$

where $N = \frac{L}{c}$, $c$ being the circumference of the coupled wheel.

$$d_1 = \frac{P_1 n_0^2 L^3}{48EI} \times \frac{2L}{3c} \quad . \; . \; . \; . \quad (3)$$

*Impact formula.* — The deflection $d$ due to an uniformly distributed load $p$ per foot run is

$$d = \frac{pL^4}{64EI} \quad . \; . \; . \; . \; . \quad (4)$$

The impact represented by the relation $i = \frac{d_1}{d}$ will be :

$$i = \frac{P_1 n_0^2 L^3 \times 2L}{48EI3c} \times \frac{64EI}{pL^4}$$

$$i = \frac{8P_1 n_0^2}{9cp} \quad . \; . \; . \; . \; . \quad (5)$$

and

$$n_0^2 = \frac{p}{(w+p)\,d} \quad . \; . \; . \; . \quad (6)$$

and finally

$$i = \frac{8P_1}{9c\,(w+p)\,d} \quad . \; . \; . \; . \quad (7)$$

This expression is the impact due to a single engine with *Hammer blow* P.

As a per cent, the impact

$$= \frac{1\,050\,P_1}{(w+p)\,cd} \quad . \; . \; . \; . \; . \quad (8)$$

wherein

$P_1$ = *Hammer blow* at one revolution per second :
$w$ = dead weight per foot run ;
$p$ = live load per foot run ;
$c$ = circumference of the driving or coupled wheel in feet ;
$d$ = static deflection in inches due to the applied load.

This expression shews that the impact is inversely proportional to the total loading of the bridge, to the circumference of the wheels and to the static deflection : this last factor shews that the impact is proportional to the square of the frequency of vibration (when considering the total force acting).

The deflection is generally a definite factor of the span so that the impact diminishes as the span increases.

The tests shewed close agreement between the calculations and the test figures.

*Compound and electric locomotives.* — As these engines are in balance the impact formula given above can not be used when dealing with them.

In service, however, these engines also cause a certain impact effect : a periodic force like that due to the centrifugal force caused by the supplementary counterbalance weights should result. The tests made by the American Railway Engineering Association shew that the impact with these engines is about a third of the ordinary. This conclusion indicates that for broad gauge engines $P_1$ will never be less than 0.2 ton.

A comparison of the calculated and measured results shews agreement for long spans, on which the critical speed is reached and also for short spans. In the latter case it is not therefore a question of synchronisation but of excessive vibrations of another kind.

For double headed trains, the value of the constant $i$ ought to be increased. If two locomotives of the same type give rise to synchronised vibrations, the effect on bridges of spans of 120 feet or more will be doubled.

For spans exceeding 120 feet :

$$i\,\% = \frac{2\,100\,P_1}{(w+p)\,cd}$$

and for spans less than 120 feet :

$$i\,\% = \frac{1\,050 P_1 \left\{1 + \dfrac{\text{span} - 30}{90}\right\}}{(w+p)\,cd}$$

If the driving and coupled wheels of the two engines were of different diameters, it is reasonable to suppose that the simultaneous effect would not be greater than with a single locomotive. This point may require to be investigated further.

The impact measured in the tests was that obtained with the test locomotive which owing to its design might give high impact values. It would be incorrect to consider this impact as applying to the maximum rolling load used in the calculations. The maximum rolling load might correspond to the case of two well balanced engines when the impact would be smaller. This point must not be lost sight of.

Taking a particular instance from India of a goods train drawn by two 2-8-2 locomotives, with a maximum axle load of 17 tons, the load hauled being 1.35 tons per foot run, the terms of the impact formula would have the following approximative values :

$w + p_{max} = 3.0$ for nearly all spans.

$$P_1 = 0.604$$

$$c = 14.8 \text{ feet}$$

$d_{max}$ taken as $\dfrac{1}{2\,000}$ of the span $= \dfrac{L \times 12}{2\,000}$

when

$$i\,\% = \frac{1\,050 \times 0.604 \times 2\,000}{3 \times 14.8 \times L \times 12} = \frac{370}{L}.$$

After having compared the results obtained by the Indian Railway Bridge Committee, the American Railway Engineering Association and the Great Indian Peninsular Railway, the Commission came to the conclusion that the general impact formula could be written as

$$I = \frac{65}{45 + L}$$

wherein L is the length under load in feet.

*Other impact causes.* — The Commission considered the speed itself to have little effect. Defects in the track have no real influence except on parts of the flooring and on short spans. Wheel flats have less effect than is generally supposed, especially as regards impact. Any synchronism is much less harmful than

that resulting from the counterbalancing of the reciprocating parts.

(*Signed*) H. N. Colam, F. Harrington, L. H. Swain,
*Secretary*, Muzaffar Hussain.

---

## Appendix A.

### *Method of calculating the Hammer blow.*

The centrifugal force due to the excess counterbalance weight required to balance the reciprocating parts has been expressed by $P_1$ for the whole of the wheels at a speed equal to one revolution per second. $P_1$ can be calculated if the equivalent out of balance weight M at the crank pin representing a certain proportion of the weight of the reciprocating parts on one side of the locomotive is known.

The reciprocating parts are :

Piston and piston rod;
Crosshead;
Small end of the connecting rod.

If $r$ be the crank radius in inches, the centrifugal force of the equivalent weight M on the crank pin is :

$$\frac{M 2\pi r()^2}{gr 12} = \frac{\pi^2 Mr}{g \times 3}.$$

This expression gives the value for one side of the engine. Since the weights on the other side are at 90° the resultant effect from both sides is :

$$\sqrt{2} \times \frac{\pi^2}{g \times 3} \times Mr = 0.145\ Mr \text{ in lb.}$$

and in tons

$$P_1 = \frac{0.145 Mr}{2\,240} = 0.000065\ Mr.$$

---

## Appendix D.

### *Impact on multiple track bridges.*

a) *New bridges.* — Up to the present the tests have been carried out with loads on one line only, and there is no information available for bridges with trains on all tracks at the same time.

The impact formula for a single line is $\frac{65}{45 + L}$. This formula applies to the case of a double headed train, the two engines being in synchronism. When there are several lines on the bridge it is most improbable that the condition of synchronism would occur on them all. Furthermore, in the case of two tracks, it is hardly likely that four locomotives would be in exact synchronism of speed, counterbalance weight phase, time at which they come on to the bridge, etc. It is therefore logical to expect a reduction of the impact effect for bridges with several tracks. In the B. E. S. A. specification the co-efficient is of the form :

$$I = \frac{120}{90 + \frac{n+1}{2} L}$$

where $n$ is the number of tracks. For short spans the reduction given by this formula is too small.

The Committee suggests the following formula for the *main girders alone :*

$$I = \frac{65}{45n + L}.$$

The total impact for one single track must be taken into account for the details of the structure.

b) *Existing bridges.* — The basic formula is :

$$i\,\% = 2 \times \frac{1\,050 P_1}{(w+p)cd}$$

two trains in synchronism on two tracks, or one train double headed on one track being taken into account.

---

## Appendix E.

*Secondary critical speed.*
*1921 tests of the Indian Railway Bridge Committee.*

Mr. Lloyd Jones has observed that in certain large bridges, such as the Adamwahan Bridge, 257 feet long, and the Kotri Bridge, 358 feet long, at double the critical speed appreciable vibrations though of smaller magnitude than those at the critical were produced.

The diagrams shew clearly that these cumulative vibrations have the same frequency as those at the critical speed and not double it, as would be the case if there were synchronism with the counterbalance.

The bridge vibrates at its own critical speed regardless of the speed of the train; there is therefore no evidence of any second synchronisation.

The explanation of the phenomenon is that at every second revolution the bridge receives a blow, causing a vibration which is not completely damped out by the succeeding one, and which becomes to some extent cumulative.

The effects never become as great as at the critical speed. The Committee is of the opinion that this synchronism at double the critical speed does not occur in practice, and that the impact should be calculated at the first critical speed.

*Report defining the work to be done by the Bridge Sub-Committee.*

The Indian Bridge Committee in 1917 carried out valuable and important investigations on the vibration of beams. The results were considered incomplete and further enquiries were judged necessary. The new Sub-Committee was formed for this purpose.

The report Mr. Lloyd Jones made on his return from America gives the opinions of American engineers on these questions. These engineers may be divided into three classes.

Some of them, practical, busy men, apply generally the Pencoyd formula, which has always given good results. They agree that it leaves something to be desired, and that further investigations might improve it : they are not particularly anxious for alterations which, in practice, are not demanded.

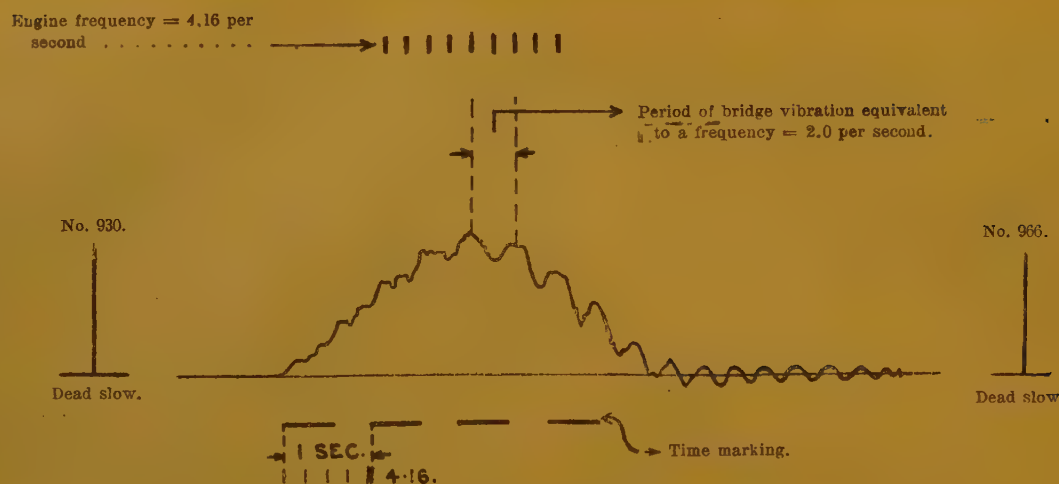
Then there are the professors, such as professor Turneaure, who have less practical experience than the former and are most interested in the mathematical solution of the problem.

The last group includes practical engineers, notably Mr. Gustave Lindenthal, considered to be the best bridge engineer in the world, Mr. Reichmann and Mr. Richard Khuen of the « American Bridge Company », and Dr. Waddell, who consider that further tests are not needed.

Mr. Lindenthal points out that all the metal bridges built have been designed in accordance with the most widely varying hypotheses as to rolling loads, wind, etc., and that consequently the numerical influence of the impact coefficient is very small if it is only a question of varying the permissible working stress shall we say between 26 and 28 % of the elastic limit. His opinion appears to be largely influenced by the fact that throughout the world there are numbers of bridges designed without any impact allowance or with different impact allowances, *all standing up well and giving good service.*

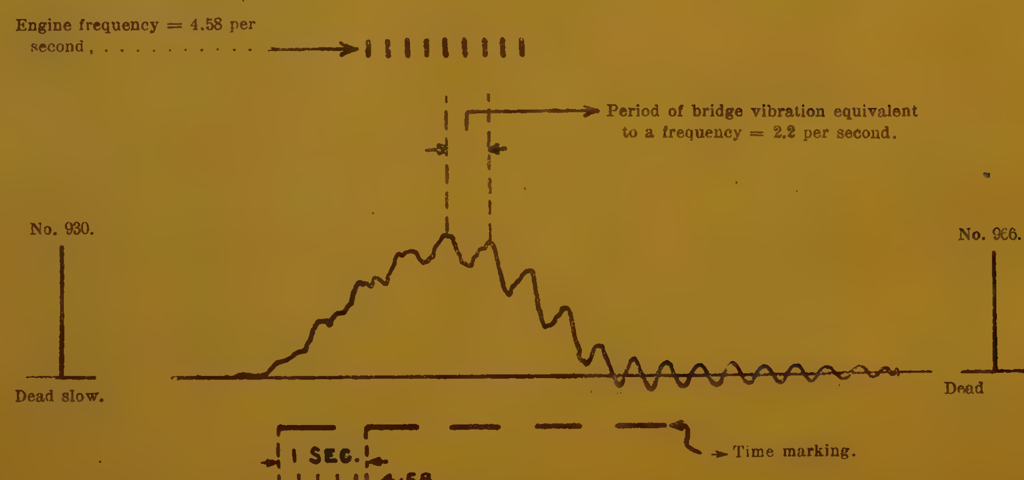
In 1892 the Indian Government laid

Figs. 11 to 14. — Typical deflectometer diagrams at second critical speed.



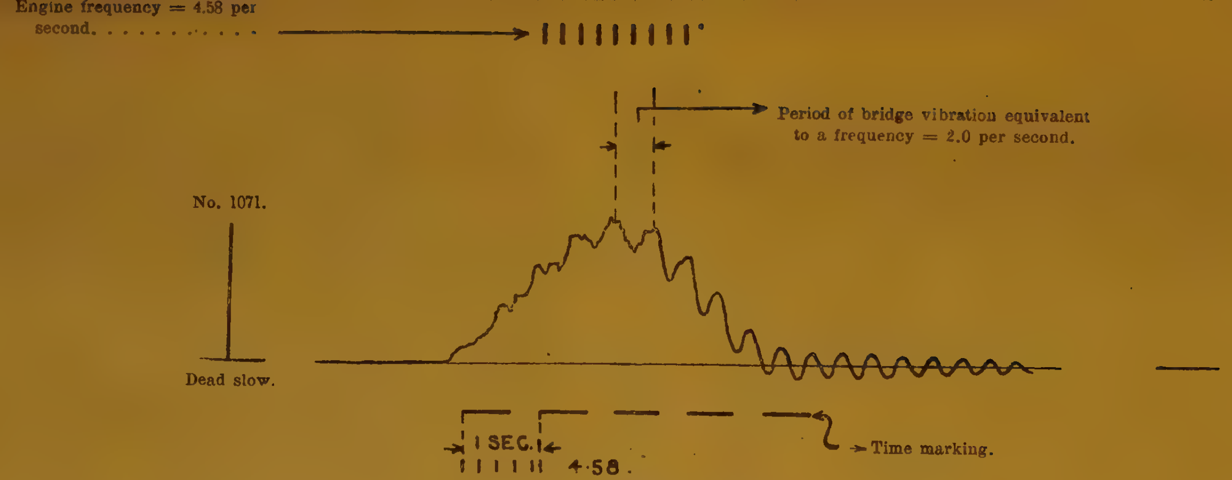

257-foot span.

Fig. 11. — Indian Railway Bridge Committee. — Test No. 958.



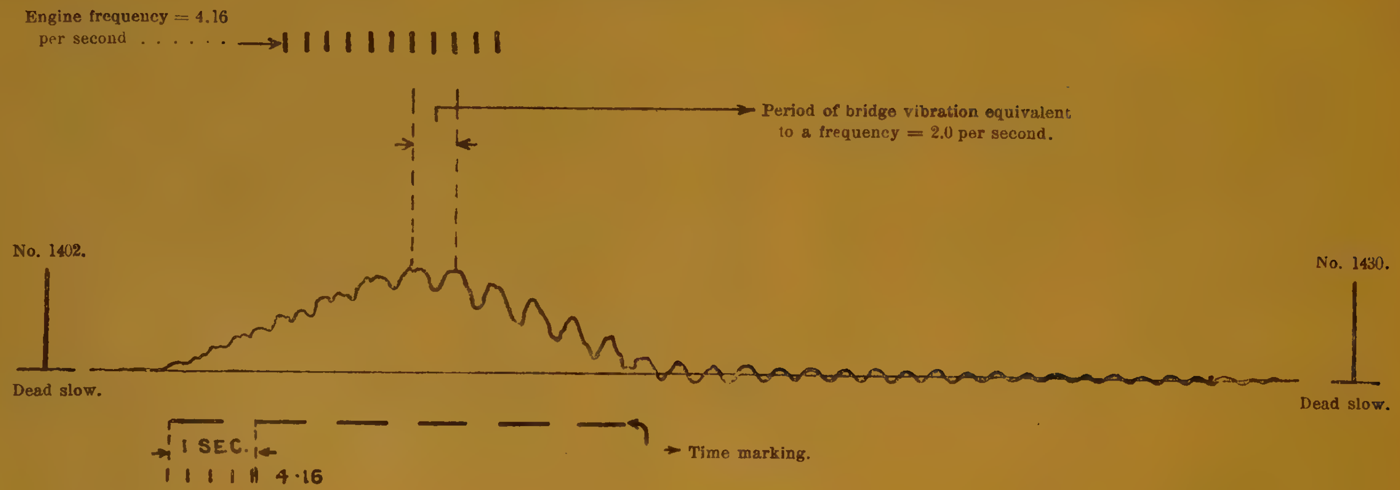

257-foot span.

Fig. 12. — Indian Railway Bridge Committee. — Test No. 962




257-foot span.

Fig. 13. — Indian Railway Bridge Committee. — Test No. 1067.




358-foot span.

Fig. 14. — Indian Railway Bridge Committee — Test No. 1426.

down rules for dealing with impact. It noted that the co-efficient probably varied for each member of the same bridge and for different locomotives and speeds. It was even suggested that to some extent it might vary with the temperature. Obviously a compromise had to be come to between the different requirements.

The impact formula used so far gave exaggerated results. The reason is that high coefficients calculated for lighter, but less well balanced, locomotives have been used in connection with heavy loading.

The Commission will see if a saving of not less than 15 % of the weight of the structure can be effected; otherwise it would be useless to set aside the Pencoyd formula.

### *What impact formula should be used in future?*

The fundamental cause of impact is the horizontal balancing of the inertia forces due to the reciprocating parts, which requires the use of supplementary counterbalance weights, and these give rise to periodic centrifugal forces of considerable magnitude at high speeds.

The value of the impact is difficult to determine in the case of long spans owing to the interference of the vibrations due to the successive blows from the different driving or coupled wheels.

The important point is not so much to determine the highest value of this quantity, but its practical maximum value. The difference between the two can be covered to a large extent by the co-efficient of safety used in the calculations. The formula to be used must also be a practical one and contain as few variables as possible. In practice the impact depends not only upon the engine and its balancing and speed, but also on all the parts of the girder, weight, moment of inertia, etc. It would be of little use to take account of all these factors as the span is the most important variable.

Another point is also under discussion, namely that the maximum impact for any individual member does not necessarily occur when this member is most highly stressed by the static load. This factor need not be a maximum for all parts at the same time. It should be sufficient to remember that the maximum strain at a point in shear does not occur under the same loading as under bending moment.

To lessen the harmful effects of impact due to excess counterbalance it is desirable when building new 2-cylinder locomotives to limit the possible dynamic increase of the wheel loads. This dynamic increase should not exceed for the whole of the coupled wheels 50 % of the static load on the driving wheel.

It is desirable also that locomotives badly balanced should be scrapped without hesitation.

### *Notes of the Director of Civil Engineering at the opening meeting of the Sub-Committee.*

1. — It must not be forgotten that the question of impact should be considered from a *practical point of view* leading to a reduction in the weight of material in railway bridge structures.

The practical point of view is notably to consider *real trains* running at *working speeds*. For example, a heavy goods train cannot run much faster than 30 miles an hour on bridges of 150 feet or more.

2. — There is already a large enough number of test reports. Possibly they

only require to be classified and studied. It would be useless to make a large number of further tests.

3. — The maximum impulse due to the excess counterbalance may be described by the expressive term « *Hammer blow* ». It would be well to establish its value at the *practical maximum working speed*, and at the *critical speed*.

For goods engines, it is of little use to exceed a speed of 35 miles an hour; for passenger locomotives the maximum speed is 70 miles an hour.

According to professor Inglis the additional deflection should be proportional to the ratio of the Hammer blow to the total load on the girder.

Added deflection :

$$d_1 = \frac{2Pl^3}{\pi^4 EI} 2Nn_0^2$$

$$n_0^2 \approx \frac{1}{d} \approx \frac{EI}{Wl^3}$$

$$d_1 \approx \frac{P}{W} \approx \frac{\text{Hammer blow}}{\text{Total load}}.$$

4. — It is important to realise that the impact co-efficient is a *percentage of the live load*. If the live load under test is less than the maximum loading the bridge may have to carry it is of little importance if the impact is high. It would be a mistake *to apply a high impact co-efficient so obtained in the maximum loading of the bridge*.

L. E. Hopkins, 1925.

---

[ 625 .13 (.44) ]

# Note on the opening out of the Batignolles tunnels,

By Pierre LÉVY.

Figs. 1 to 37, pp. 395 to 431.

(*Proceedings of the « Société des ingénieurs civils de France »*, May-June 1926.)

The opening out of the Batignolles tunnels, commenced at the end of 1921 and now nearly finished, is of interest owing to its position in Paris and to the difficult nature of the work. A clear account of the work will be given below commencing with a reference to its bearing on the general re-arrangement of the lines between Asnières and Paris approaching Saint-Lazare Station, carried out at the same time. After describing the scheme as a whole and in its sub-divisions, particulars will be given of the more important methods of carrying out the work chosen, with any special features that choice presented, and in addition the reasons why, after a study of the problem, the various methods adopted were selected.

The Engineers concerned found the preliminary investigations of great interest.

## I. — Present position — Proposed arrangement. Object of the undertaking.

The diagram, figure 1, in the upper figure shews the lines as they were before the tunnels were removed, and before the first section of the suburban lines on the right-hand bank of the river were electrified in April 1924; the lower figure shews the situation as it will be when the opening out of the tunnels has been completed.

The old layout was as follows :

Saint-Lazare station : 27 roads.

From Saint-Lazare to Cardinet, 4 double tracks (8 roads) that is, the Auteuil lines, the Versailles, Les Moulineaux, Saint-Nom-la-Bretèche lines, the Saint-Germain and Argenteuil lines, and finally the main lines.

Beyond Cardinet there were three double tracks (six roads) namely, the last three quoted above.

In addition, two lines, known as auxiliary lines, branch out to the right at the Asnières end of the tunnel, and connect the station with the carriage sidings at Clichy, where the main line stock is sent for inspection and cleaning.

The new lay-out is very different. One of the essential principles in electrification is to reduce, as far as possible, the number of points and crossings, and thereby sections used by different services : a separate pair of roads to Saint-Germain and another to Argenteuil had to be provided.

These lines were needed for the reason just given, and because one pair of lines would have been insufficient to handle the estimated number of electric trains, which will be considerably more than the number of steam trains, and the still greater number during the transition period of both steam and electric trains.

These important auxiliary lines crossed the main lines just outside the tunnel before it was demolished. They can now be extended, as shewn in figure 1, almost into Saint-Lazare station, so that empty stock proceeding to or from the station will no longer interfere with the running of main line passenger trains.

These two essential results have been obtained by the Auteuil trains terminating at Cardinet and by opening out the Batignolles tunnel, advantage being taken at the same time to make the cutting as large as possible, and of considerably greater width than the total useful width of the demolished tunnels.

Although traffic requirements will be met for many years, the work done forms part only of a larger scheme of improvement which may include the construction of an underground station or some other alternative scheme thereto (1).

When opening out the Batignolles tunnel, other works, forming part of the whole scheme, were carried out; line by line the old lay-out has been altered to the new between Paris-Asnières-Bécon and Bois-Colombes; new platforms and buildings were built at Cardinet, Clichy, Asnières, Bécon and Bois-Colombes. At the last two places local termini have been built; eight raised platforms have been built in Saint-Lazare station, and in consequence the signalling as a whole has been considerably modified. It may be of interest to mention this point, because in many ways the progress of the opening out of the tunnel closely depended on the state of these other works, and this sometimes made the carrying out of the work as a whole difficult.

Returning to the tunnel itself, all that has been said may be summarised by noting that the opening out of the tunnel had become necessary to enlarge the « Batignolles bottle neck » : the considerable gain in width has been obtained by setting back the retaining wall of the new cutting beyond the original arches and by gaining the space occupied by the footings, as shewn in figure 2 : the total amount is 11.70 m., (38 ft. 4 5/8 in.) which in Paris may be considered as of considerable importance.

(1) As a note dealing with another important improvement which has already been made in the Paris suburban system, the diagrams in figure 1 shew the abolition of the « Asnières crossing ». On the lower diagram will be seen the flying junction which has replaced the crossing of the Poissy-le-Havre main lines and the Argenteuil line; this arrangement carries the former of these two groups over the latter (the latter being electrified). This alteration, which gives considerably greater safety, was brought into operation on the 28 August 1923.

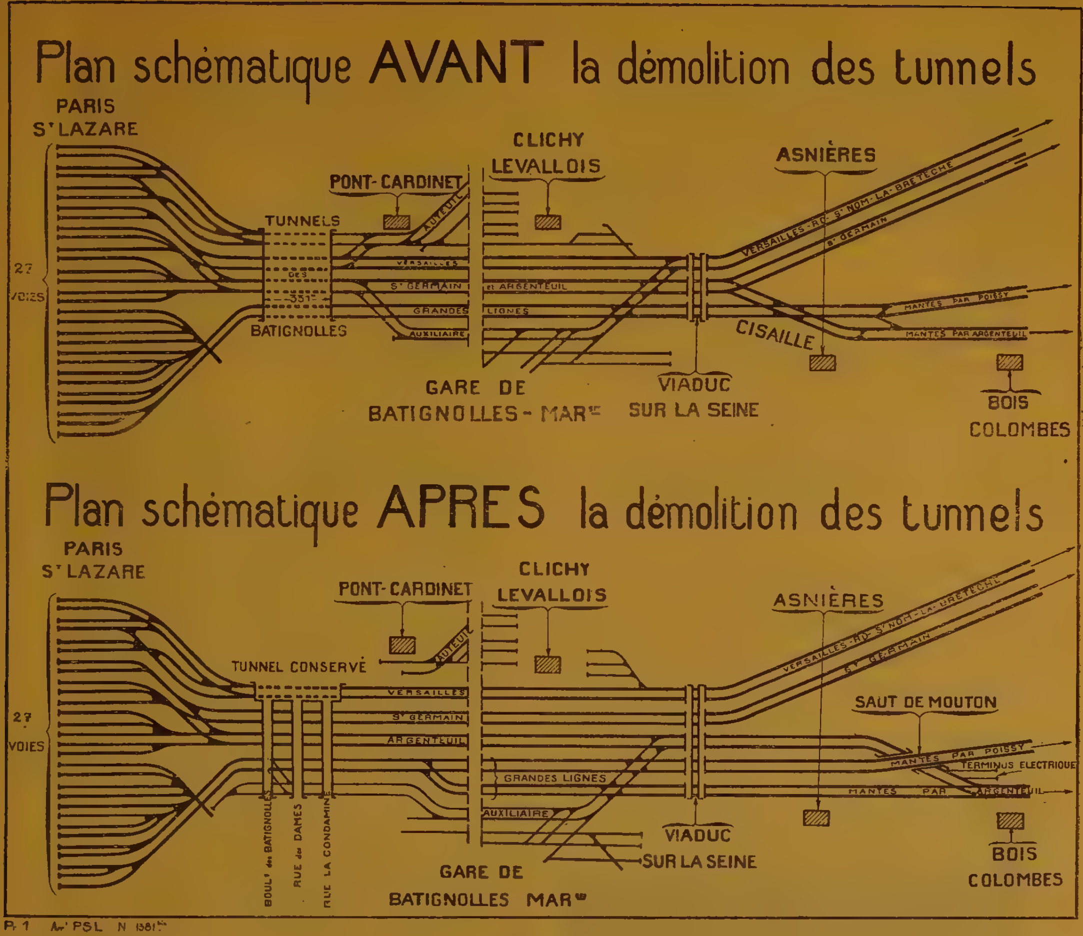



Fig. 1.

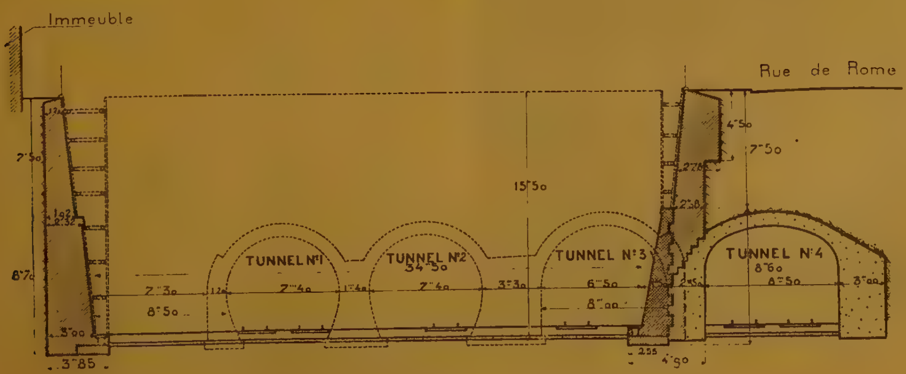



Fig. 2. — Transverse section of the cutting.

*Explanation of French terms in figures 1 and 2*: Auxiliaire = Auxiliary line. — Cisaille = Crossing. — Gare de Batignolles Mar^ces = Batignolles goods station. — Grandes lignes = Main lines. — Immeuble = Building. — Plan schématique avant la démolition des tunnels = Diagram of lay-out before opening out the tunnels. — Plan schématique après la démolition des tunnels = Diagram of lay-out after opening out the tunnels. — Saut de mouton = Flying junction. — Tunnel conservé = Remaining tunnel. — Viaduc sur la Seine = Viaduct over the Seine. — 27 voies = 27 platform roads.

The opening out of the tunnel was also considered to be necessary for reasons of safety, as shewn by the serious accident of 5 October 1921, aggravated by the gas lighting then used, but since superseded on the State Railways by electric lighting.

**II. — General description of the tunnel. — Sub-division of the work. — Guiding principles and stages of the work.**

The length of the tunnels is 330 m. (1 082 ft. 8 in.).

Tunnels 1 and 2, that is to say, the two left hand tunnels in figure 2, dated from 1842, and were amongst the last traces of the first railways built in France; tunnel 3 was built in 1865, and 4, in 1910.

The original lay-out is clearly shewn in the upper view of figure 3, and in the lower, the new arrangement. The eight roads in the cutting, in addition to the two roads in the retained tunnel, No. 4, are shewn as well as the three road bridges : Rue La Condamine, Rue des Dames, Boulevard des Batignolles, crossing the cutting.

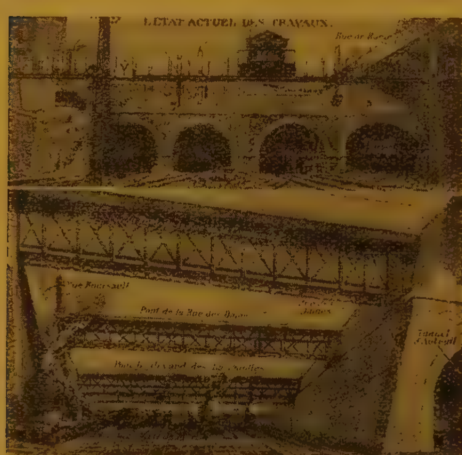


Fig. 3. — Appearance of line before and after opening out the tunnels.

The change over step by step from the old conditions, shewn in the upper part of the photograph, to those in the lower, had to be made without interfering with the running of the trains. A cutting had to be provided in place of the opened out tunnels.

The proximity of the Rue de Rome on the right hand side, and of important buildings on the left, made an earth cutting with the proper slope impracticable, and almost vertical retaining walls had to be built.

The first stage of the work was therefore to build these walls, shewn in section in figure 2, and containing 15 000 m³ (19 620 cubic yards) built in timbered trenches.

The second stage was the removal of the spoil between these walls and the opening out proper of the tunnels themselves; at the same time the Rue La Condamine and the Rue des Dames bridges were constructed.

The third stage, begun towards the end of the second, included the building of the Batignolles bridge, which is in fact three bridges side by side, of 40 m. (131 ft. 2 3/4 in.) span, details of which will be given later.

The fourth and last stage, now in hand, is the opening out of the last section of the tunnel beneath this Batignolles bridge, which could not be done until this bridge had been completed, and will complete the undertaking.

Details of each of these stages are given below.

*First stage.*

The wall on the left-hand side facing Paris, shewn in section, figure 2, is 17 m. (55 ft. 9 1/4 in.) high; 1.70 m. (5 ft. 6 15/16 in.) thick at the top, and 3 m. (9 ft. 10 1/8 in.) at the bottom, and at the level of the reinforced concrete footing 3.85 m. (12 ft. 7 1/2 in.) wide. The ground — hard marl, layers of gravel, etc. — was good, and a fairly thin section could be used, based on a natural

earth slope of 60°, certainly less than the angle of repose would have been. The foundation, of Beauchamp sand, is excellent; it was known it would be, from other excavations of equal depth.

Particular care was taken in timbering these deep trenches owing to the closeness of lofty and important buildings, and as result no settlement occurred.

The right-hand wall was another matter.

The lower part is formed by the wall between tunnels 3 and 4; tunnel 4 was retained as it is situated under the Rue de Rome, and its opening out would have very considerably increased the cost, and would have seriously inconvenienced the inhabitants thereof during the work. However, it now runs along the side of a large cutting with many openings between the two, so that the ventilation has been improved, and ways out in case of accident secured.

The common side wall of tunnels 3 and 4 has therefore been retained, but its section would have been quite insufficient, as shewn in figure 2, to take the thrust exerted by the tunnel roof, and the earth above; it was therefore necessary to build into the foot of the wall the reinforcement shewn cross hatched in figure 2 which, as described later, was found to be very difficult.

Finally, the appearance of the walls when completed was not overlooked. The lower part was faced with coloured stone, the top with hard white stone of much smaller size, a course of hewn stone setting off the difference between the two, the lower portion giving in particular an impression of strength.

Masonry was chosen instead of reinforced concrete after a close examination of the various possible ways of building these walls. The whole of the work involved, the opening out the tunnels and the construction of the three bridges, was, in 1912, the subject of a competition for which various solutions were received; some of the competitors actually proposed to construct the walls in reinforced concrete. In this case, as the ground was good, a wall of reasonable thickness, without the help of reinforced concrete, could be used, and it appeared an advantage to use a material which could without difficulty be built directly up against the earth in position without disturbing its equilibrium; these considerations were of particular importance on the side above the remaining tunnel 4.

It may be added that the walls would hardly have looked as well in reinforced concrete as in masonry.

### *Second stage.*

The second stage now to be considered was to remove the spoil and open out the tunnels, except under the Boulevard des Batignolles, and was the main part of the undertaking.

The job could be handled in several ways.

Ignoring such methods as moveable centering, of which more will be said later, two methods naturally suggested themselves : to demolish the linings and let the tunnels fall in and then remove the spoil; or else deal with them as they were, in which case centering would have to be put in place in the tunnel, and this could only be done by stopping the running of trains through the tunnel during the operation.

Obviously the most economical was to allow the lining to fall in, hence the temptation to use this method for the three tunnels.

This method was at once precluded by the fact that the wall between tunnels 1 and 2 was too weak (fig. 2) to ensure that the destruction of one tunnel would not bring down the other, the more so as the masonry of this thin wall was thought to be in poor condition.

Under these conditions traffic would have had to be stopped on four roads and not on two, and this could not be allowed.

It was therefore decided to take down the linings of tunnels 1 and 2 on centering, which would not disturb the stability of tunnel 3, and then to allow this last tunnel to fall in. Figure 2 shews the heavy side wall, which was of sufficient strength for this to be done, that is to say, to let the linings of tunnels 1 and 2 be removed without causing the roof of tunnel 3 to fail in.

It may be stated that technically there was not so much difference between the methods as this description seems to imply. The work might have been begun by shoring up tunnel 2 by shores in tunnel 1, and demolishing the roof of tunnel 1, thereby dispensing with the centering, but this idea was quickly abandoned for various reasons, especially by the danger of the struts being broken by material falling from the arch and burying them.

Another consideration to be taken into account was the habitual carelessness of suburban travellers. Centering, even if only 18 cm. (7 1/16 in.) thick, under the arches, increased, though only slightly, the danger to passengers on the top standing up in the tunnels. The various operations were therefore so arranged as to avoid these additional risks during the second stage of the work, in the way explained later. In consequence there was some difficulty in co-ordinating the work at the top and below.

Figure 2 shews that if the earth against the right-hand wall above the roofs was removed before the foot of the wall had been strengthened, there would have been a risk of this wall being pushed over; to carry out this reinforcement it was necessary to do away with one of the lines through the tunnel; on the other hand, the placing of centering, would have required the temporary closing of two further lines, but only two could be spared from traffic.

These requirements were not absolutely contradictory, but the only admissible solution was narrowly limited by certain conditions which could not be ignored.

This is shewn by the figures given.

First stage (fig. 4). — The trains on the Auteuil line (group I) which formerly passed through tunnel 4, were terminated at the Cardinet bridge, and the three pairs of lines remaining in service, groups IV, III and II were slewed one space towards the right from their normal position. Tunnel I being free, a temporary line was laid through it and centers put in. As regards the earthwork, sufficient earth was left in place to support the right-hand wall, as yet insufficiently strong to stand by itself, as already explained.

Second stage (fig. 5). — The main lines, group IV, were relaid under the centering put in the tunnel, as no double deck coaches were run over these lines, the suburban trains, in which they are used, running through tunnels 3 and 4; but for the reason given above no further progress could be made with the earthwork.

Third stage (fig. 6). — During this stage, the side wall between tunnels 3 and 4 was strengthened. As it was undesirable to run the suburban trains of group III through tunnel 2, with its roof supported on centering, with the tracks in their position, this pair of lines was divided, the left-hand road alone being laid as a single line through tunnel 2; being in the middle of the tunnel there was such clearance to the centering that no danger would occur to careless passengers; the right-hand track was retained in tunnel 3.

Leaving this line open to traffic through the tunnel during the strengthening of the side wall caused much in-

convenience on account of smoke and the restricted space available for the contractor, but it was considered to be desirable, and later was found to have been essential.

During this stage also, no further progress could be made in the earthwork, for the reason already given.

FOURTH STAGE (fig. 7). — The work could now be carried on more freely; the whole of the excavation was removed, largely by means of a temporary line, seen on the left of the figure. The roofs of tunnels 1 and 2, supported by centering, were demolished.

FIFTH AND LAST STAGE (fig. 8). — Demolition of tunnel 3 by allowing it to fall in and making good the wall at the spring of the demolished arch.

NOTE. — At the beginning of paragraph II mention was made of a moveable centering. During the competition of 1912 a suggestion was made to demolish the linings by a moveable centering, consisting of a steel framework travelling above the tunnels on a track carried by the side walls, the framework carrying centering several metres in length like a C. to support the lining in front of the working face during demolition. This method of working, sound in itself, would not, however, have given the same absolute guarantee afforded by continuous centering, in view of the lack of knowledge of the quality of the masonry and the distance to which a crack starting at the working face might eventually extend. By reason of the capital importance of avoiding any accident, continuous centering was preferred.

## III. — Details of the work of opening up and of excavation.

The proposed scheme was as described above, and was followed exactly.

Certain interesting features were noted during its execution.

*Centering.* — Ordinary rolled joists 18 cm. (7 1/16 inches) deep bent to the profile of the lining were used. Figure 9 shews them seen in perspective after the demolition of the roof.

They were placed 0.50 m. (19 5/8 inches) apart (1).

Each joist was joined by six bolts (some of which are seen in figure 9) to the joist on each side of it.

Boards 27 mm. (1 1/16 inches) thick bearing on the lower flanges formed a continuous shuttering without any projection or cavity on the inner face of the centering (the boards being suitably chamfered). Finally, a layer of sand was placed between the lining and the boards so that the arch was effectively supported, this sand being consolidated by a little lime, a device proposed by the contractor, and found of considerable utility during the demolition as a good support for the workmen.

The centering had to be placed with exceptional care, as its inner face was actually within 5 cm. (2 inches) of the structure gauge, and about 15 cm. (6 in.) from the extreme parts of the double deck coaches: in each case a careful check was therefore made by means of a structure gauge in wood carried on a special vehicle, the axleboxes of which were blocked so that the clearance should not be affected by the deflection of the springs. Lead fingers were fixed on this gauge, and after passing through the tunnel, an examination was made to see if any were bent, which would have shewn insufficient clearance; in no case was this found.

*Strength of the centering.* — Each joist was calculated to carry easily the linings on the assumption that they were broken

(1) Experience shewed that they could be spaced further apart; under the boulevard they were placed 1 m. (3 ft. 3 3/8 in.) apart, and the 27 mm. (1 1/16 inch) boards were replaced by planks 4.4 cm. (1 11/16 inches) thick.

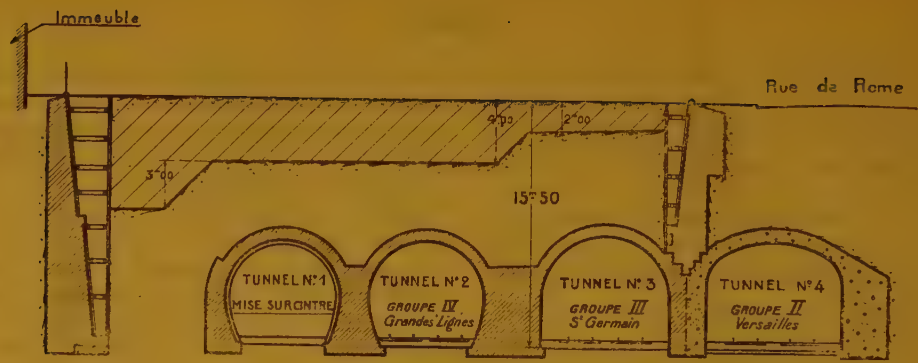



Fig. 4. — Opening out the tunnels. — First stage.

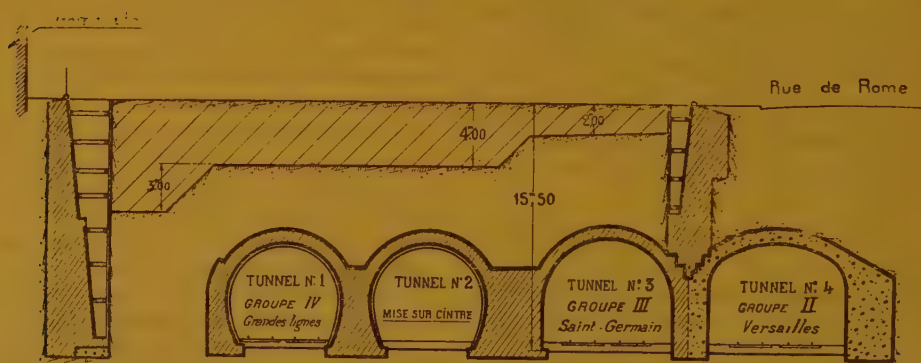



Fig. 5. — Opening out the tunnels. — Second stage.

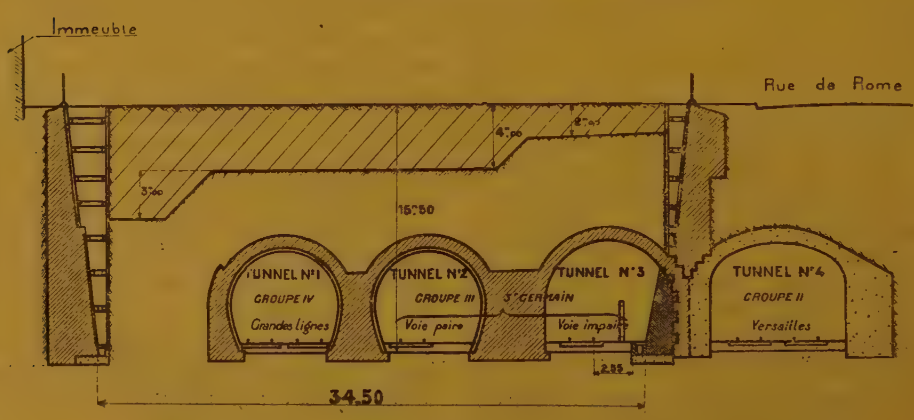



Fig. 6. — Opening out the tunnels. — Third stage.

*Explanation of French terms of figs. 4 to 8* : Groupe IV. Grandes lignes = Group IV. Main lines. — Immeuble = Building. — Saint-Germain = Saint-Germain lines. — Tunnel N° 1. Mise sur cintre = Tunnel No. 1 on centering. — Versailles = Versailles lines. — Voie de travaux = Temporary line. — Voie impaire = Down line. — Voie paire = Up line.

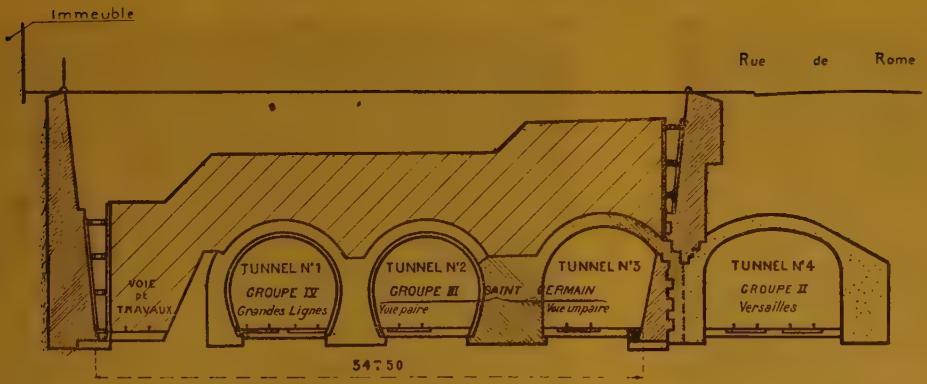



Fig. 7. — Opening out the tunnels. — Fourth stage.

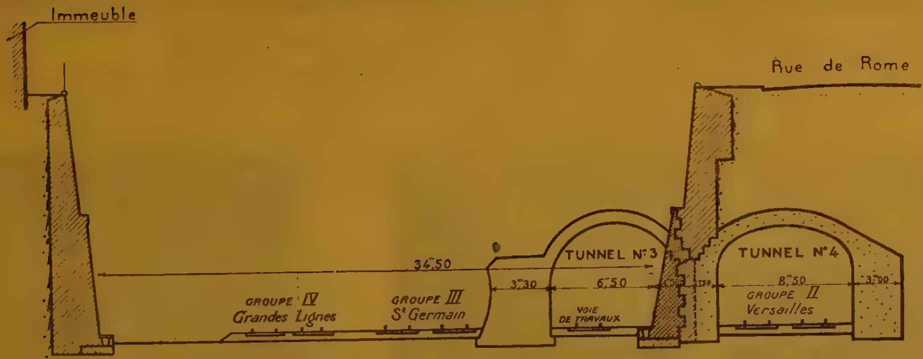



Fig. 8. — Opening out the tunnels. — Fifth stage.

at the crown or at the spring of the arch; the extra strength was not without use, as the machinery used in the excavation work was working at little distance above them.

Figure 10 shews the centering in place in the tunnels.

The centres were made in three pieces connected by fish plates : workmen are seen bolting them up. When this has been done, the centering is turned over on its end and put into position alongside those already erected.

The ends of the centerings were set in a concrete base. At the extreme ends will be noticed the angles forming the bearing.

It was found to be possible to place and set as many as 24 centerings per night.

The contractor provided a boarded floor on the wagon shewn in figure 10 on which to mix the weak concrete used for filling, the materials being supplied through a hole in the roof. On the left a workman will be seen packing this weak concrete between the centering and the lining.

Figure 11 shews the centering being taken down : four workmen moving back across the centering not yet removed are pulling towards them, with two ropes, the upper third of the centering which weighs 120 kgr. (265 lb.). The

Fig. 9. — Perspective view of centering, its erection and wooden sheeting.

Fig. 10.
Assembling and erecting centering.

Fig. 11.
Removing centering.

Fig. 13. — Bonding and reinforcement of side wall.

train service was of course not interfered with whilst this was being done, but special arrangements were made to obtain warning of approaching trains so that the centering should not be taken down whilst trains were passing.

*Work on the track between the stages.* — Diagrams for the various stages (figs. 4, 5, 6, 7 and 8) shew that between them the tracks were transferred from one tunnel to another; owing to the very heavy traffic out of Saint-Lazare only a

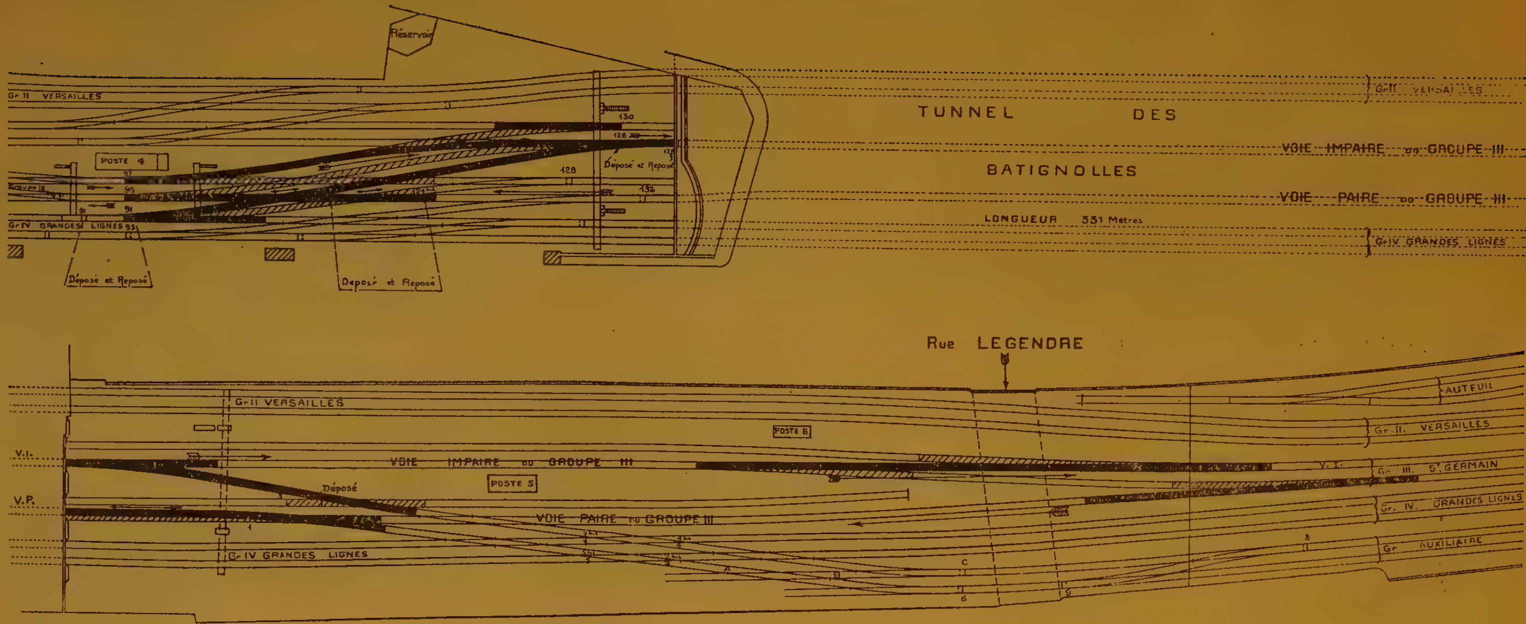



Fig. 12. — Opening out of tunnels. — Alterations to track when passing from the second to the third stage.

*Explanation of French terms* : Déposé et reposé = Taken up and re-laid. — Poste 5 = No. 5 signal box. — Voie impaire du Groupe III = Down road of Group III. — Voie paire du Groupe III = Up road of Group III.

few hours were available during the night in which to complete the whole of the alterations to the track. Especially careful advanced preparation was required between the second and third stages. Figure 12 shews, by cross hatching, the track and points and crossings taken up and, in black, the new work (the two parts of the diagram deal with work carried out at the same time; the upper in front of, and the lower beyond, the tunnel). During that night, four sets of points were taken up, two sets of points were relaid, two sets of points complete with rodding relaid, 200 m. (656 feet) of track taken up and relaid, six signals taken out, and six other signals put into service, including all interlocking.

The large amount of work done each time was the result of having to restore, not only the continuity of the lines, but all the interconnections between the various groups of lines, which are known to be very involved near Paris.

The very careful preparation for these busy nights avoided accidents, and delays to trains were prevented by the care taken in preparing for the work to be done. The workmen concerned are to be congratulated that such remarkable results were successfully repeated during each change over.

*Strengthening the side wall between tunnels 3 and 4.* — The strengthening of this wall has been referred to frequently. Figure 2 shews how it was intended to bond this masonry into the existing wall by making saw tooth shaped cuts into it. It should be mentioned that effective bonding here is essential because the lower part of the wall, in maximum vertical shear, owing to the bending moment, would be made up of the masonry centre built in 1865, the masonry (on the right-hand side) built in 1910, and the reinforcement to be built in 1923, and any slip of one part of the masonry upon another had to be avoided at all costs.

All attempts to cut into the foundation failed; even when four toothed bits driven by pneumatic drills were used it was practically impossible to touch the hard rock like surface.

Several days were lost considering what steps should be taken. After many tests made in collaboration with Messrs. Ingersoll Rand and the contractor a satisfactory solution from the point of view of the strength of the bonding was found, and one readily carried out. It is shewn in figure 13; bars 36 mm. (1 3/8 inches) diameter, each forming a strong attachment, were inserted 50 cm. (19 5/8 inches) apart and inclined at an angle of 45°. It is well known in practice that to resist sliding motion along a plane, the provision of bars of this kind inclined at 45° to the plane forms the most effective device.

The free portion of these bars was embedded in the concrete reinforcement, which also contained a number of longitudinal rods provided to tie together the successive transverse sections. A masonry facing similar to that originally intended completed it.

The reinforcements and anchorages, after some preparatory work, were carried out rapidly, the slope of the rods not causing any further difficulty, provided the special tripods supplied were available. The cost was 20.50 francs per anchorage.

The material was brought down through the hole cut in the roof of tunnel 3, shewn in figure 15.

This work, carried out in the smoke of trains in a very restricted space [2 m. (6 ft. 6 3/4 in.) deep] was one of the most unexpected and most interesting, in view of its difficulty, of the jobs which occurred during the opening out of the tunnels.

### Mechanical devices used during the opening out.

Pneumatic drills, weighing 32 kgr. (71 lb.) obtained by the contractor from Messrs. Ingersoll Rand, were largely used. Figure 11 shews a workman using one of these drills to break up the remainder of one of the side walls. The working pressure was 5 kgr. per square centimetre (71 lb. per square inch); compressed air was taken from the Compagnie Parisienne de l'Air Comprimé, from the main along the Boulevard des Batignolles, the average consumption of compressed air being 400 $m^3$ (523 cubic yards) per drill per eight-hour shift. As many as 15 drills were in use at a time. From 2 to 5 $m^3$ (2.6 to 6.5 cubic yards), according to the hardness of the masonry, were demolished per machine per eight-hour shift.

Amongst the precautions taken during the demolition of the roofs, it may be mentioned that any unequal pressure on the centering was prevented by first cutting a longitudinal opening along the summit of each of the arches 1 and 2 : the demolition was then carried out symmetrically downwards along the centerings, and wherever possible, simultaneously on tunnels 1 and 2.

### Demolition of roof of tunnel 3.

Figure 14 shews this operation, which consisted in cutting through the roof and allowing these sections of the linings to fall to the ground. The broken lines marked on this photograph shew the steps taken by the contractor to do this and avoid accidents. After making transverse cuts, the longitudinal cuts, shewn by the broken lines, were made, and when the length carried on the side wall became insufficient to support it, the arch fell in. At the point marked A on the photograph, careful inspection will shew the undercutting in question.

*Earthworks.* — The upper cut of the excavation was carried out by using the powerful excavator shewn in figure 15, weighing 60 t., with a pull of 18 t. at the crane hook.

The tunnels were found to be in good condition, but as it was not safe to assume this previously owing to their age, and the long continued corrosive action of the smoke on the 1842 mortar, a mattress of earth, about 2 m. (6 ft. 6 3/4 in.) thick was retained under the heavy steam navvy.

The contractor later on used a smaller machine (22 t., 6 t. at the crane hook) which took up less space and which was used almost down to the linings of the tunnels.

Figure 16 shews the general appearance of the excavation, with the steam navvys, cranes and lorries.

Finally, the contractor installed special equipment for removing the spoil from the wedge shaped excavation between the Rue Boursault retaining wall and the first tunnel.

Figure 17 gives a diagram, and figure 18 a photograph, of this equipment. In the foreground an excavator at work on the excavation is shown, the excavated earth and stones being raised by an elevator, and falling through a chute on to a chain conveyor, from which it is discharged into the wagons. In order to load the wagons uniformly and cheaply, without having to spread the soil by hand, the wagons were moved past the end of the conveyor as necessary.

The workman in figure 18 levels the excavation with his pick, but does not have to spread it.

Figures 17 and 18 shew the small steam shovel preparing a platform at an intermediate level, since the excavator obviously can only deal with the lower portion if the height of the working face is relatively small.

The unusual nature of the work made it necessary to choose the excavating

Fig. 14. — Demolition of lining of tunnel No. 3.

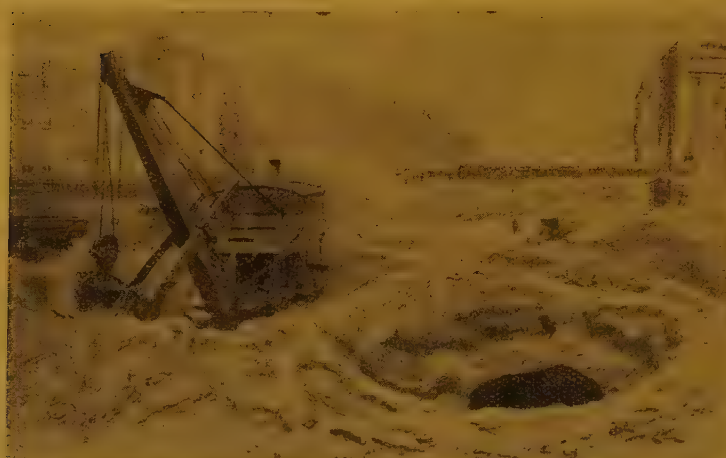


Fig. 15. — Steam shovel used on the works.

Fig. 16. — General view of excavation.

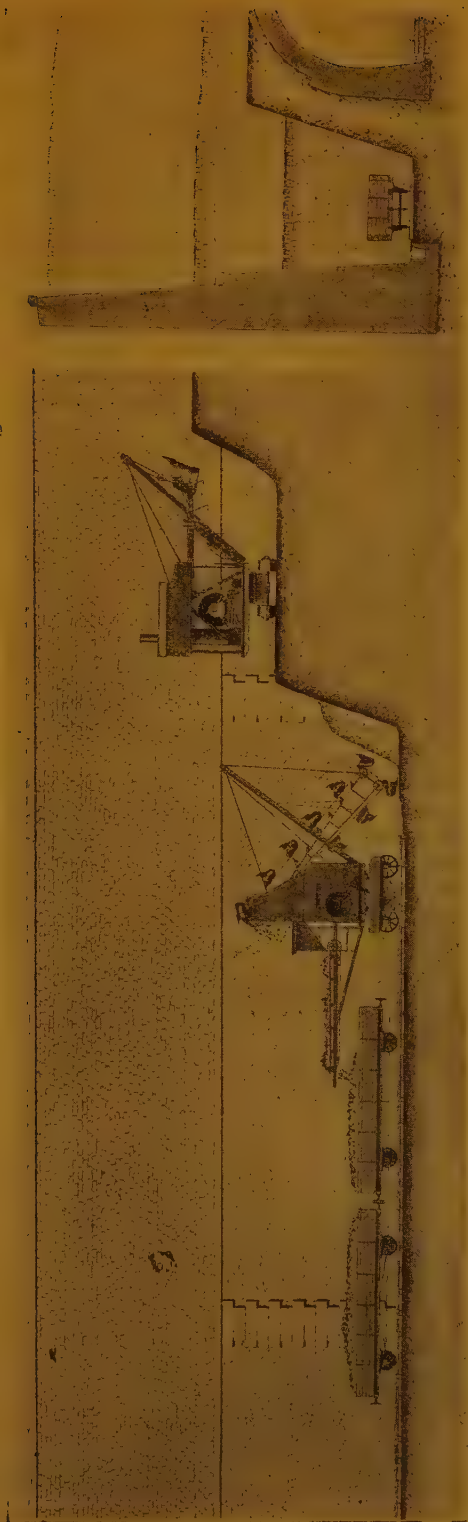


Fig. 17. — Excavating machinery used for digging out a trench along the retaining wall, Rue Boursault side.

machinery with much care, to employ very different types, and even to devise new ones. The use, as a group, of a steam shovel, an excavator below it, a conveyor, and a wagon, is quite exceptional, and is eminently suited to the work to be done.

The best output was of course obtained with the large steam navvy, the maximum output being 536 m³ (701 cubic yards) per day, and 400 m³ (523 cubic yards) over long periods.

The work was not carried out without numerous difficulties arising.

As is often the case in excavation work, disposal of the spoil was the critical point in the job. Running the lorries up an incline, making up a difference in level of 8 m. (26 ft. 3 in.) became difficult as soon as the weather was at all wet, and impossible after heavy rain, in spite of the way the incline was kept.

It was also necessary to keep open to traffic one of the two streets which cut the works into three sections, which meant leaving the ground under these streets. It was also necessary to proceed with the construction across the works of the two steel bridges which, during the excavation, were erected to carry these two streets (Rue des Dames and Rue La Condamine.)

Another difficulty was to co-ordinate the heavy traffic in the neighbourhood of Saint-Lazare with the movement of the ballast trains which carried about 41 000 m³. (54 000 cubic yards) of excavation or masonry.

After being cleared, these ballast trains did not always return to the minute in the morning : if to this is added that interchanges between the two sides across the main lines were of necessity very difficult, some idea of the minor troubles which arose from the nature of the work and from its location will be had.

Fig. 18. — Excavating machinery used for digging out a trench between the retaining wall, and No. 1 roof.

### IV. — Construction of the bridge under the Boulevard des Batignolles.

As mentioned above, the construction of this bridge formed the third stage of the undertaking, and was started towards the completion of the second stage, and in fact as soon as the completion of the Rue des Dames bridge allowed the large gas main, 1 m. (3 ft. 3 3/4 in.) in diameter, to be taken up from the Boulevard des Batignolles and built into the structure of the Rue des Dames bridge (1).

The Boulevard des Batignolles (see fig. 19) is 42 m. (137 ft. 9 in.) wide; it consists of two separate roads 12.50 m. (41 feet) wide, including the pavement, on each side of a central path 17 m. (55 ft. 9 in.) wide for the use of pedestrians. The Metropolitain Railway (« Dauphine-Nation » section via the outer boulevards) runs under this promenade above the railway tunnels. Two lines of tramway, « Trocadero-La Villette » and « Etoile-Bastille », run along one of the roads and carry a heavy traffic, and under the two roadways the following mains, shewn in figure 19, the list of which is characteristic of the problems which had to be overcome, existed:

Sewers :

2 sewers of 2.4 m. (7 ft. 10 1/2 in.) depth.

Water :

1 main 1.100 m. (3 ft. 7 5/16 in.);
1 — 0.500 m. (1 ft. 7 11/16 in.);
1 — 0.400 m. (1 ft. 3 3/4 in.);
1 — 0.150 m. (6 inches);
1 — 0.100 m. (3 15/16 inches).

Electricity :

49 important cables of the « Compagnie Parisienne de distribution d'électricité », some laid in special culverts, others simply buried;
21 cables of the « Société des Transports en commun de la région parisienne»;
6 cables of the Metropolitain;
2 pipe lines of the Compagnie Parisienne d'Air Comprimé;
2 pneumatic tubes of the Post, Telegraph and Telephone;

A number of telephone lines of the Post, Telegraph and Telephone.

Finally, a large gas main, 1 m. (3 ft. 3 3/8 in.) in diameter, one of the largest in Paris, crosses the boulevard.

The problem was therefore to construct a bridge, the upper flooring of which would carry the roads and tramways, and which would contain, within its framework, the whole of these mains as well the Metropolitain. It was also essential not to interrupt or cause any delay in the operation of any of the public services concerned, and especially the Metropolitain.

The competition held in 1912 had

(1) The Rue des Dames and Rue La Condamine bridges do not call for any special mention : the type of bridge used will be seen clearly from figure 3.

brought forward various schemes, and in particular the use of arch bridges. This idea was set aside for several reasons : in the first place it is as well in Paris to avoid structures whose strength depends upon the absolute solidity of the neighbouring ground, as important works might possibly have to be carried out later on in the sub-soil close to the bridges. Further, it was desirable to reduce to a minimum the area of the foundations so as to encroach as little as possible during the work upon the Rue de Rome, the Boulevard des Batignolles, etc.

In the third place, the arch obviously affords, for the accommodation of existing or future mains, much less space than a girder bridge. Finally, the better appearance of an arch would not have been entirely realised in the present case. If, in fact, this system had been adopted for the outer bridges carrying the two roadways, the central bridge carrying the Metropolitain would of necessity have had to be of the girder type, and the area left by the arches would therefore have been partly occupied by the Metropolitain bridge; the former bridges would not have had the expected graceful appearance. It should be added that the simple type of structure adopted in this case was particularly economical because, by reducing the supports, it allowed full use to be made of the available head room — 8 m. (26 ft. 3 in.) for a span of 43.90 m. (144 feet). Figure 19 shews the passages for the mains which are rectangular channels in reinforced concrete.

The spaces not occupied by passages contain the transverse wind bracing.

The work, as a whole, consists of three separate bridges joined together. This arrangement was made almost obligatory by the obvious necessity of constructing the bridge in stages. Further, the bridges being almost independent (only thin plates connecting them at the top) the vibrations set up in the central bridge which carries the Metropolitain are not transmitted to the side bridges nor to the mains which they carry. The separation also assists in the electrical insulation of the various mains and of the Metropolitain.

The Metropolitain is, furthermore, isolated from the railway and its smoke by a continuous brick lining, which is shewn in section in figure 19.

The roadways and paths are carried on a soling of reinforced concrete carried on the metal structure.

The weight of the structure is 1 600 t. Each of the three bridges is 15 m. (49 ft. 2 1/2 in.) wide with a skew span of 45 m. (147 ft. 7 3/4 in.), the skew angle being about 75°. On the Clichy side very heavy wings are attached, as shewn in plan in figure 20 : the left-hand wing carries part of the Rue Boursault; the right-hand wing carries an accommodation road to the Messageries building; this latter road, when loaded, weighs 1 030 t.

As in the case of the second section, it may be of interest to recapitulate the general principles which decided the various stages of construction. Needless to say, the chief difficulty was to draw up a programme for the different stages.

In the first place it was decided how to build the part of the bridge under the Metropolitain, as this was obviously the most difficult to construct, and certain features of the method chosen for carrying it out would govern the order in which the stages of the whole work followed.

The method of building the Metropolitain bridge, or rather the bridge round the Metropolitain, will now be described.

The masonry lining was first uncovered and then carried on centering, using the same steel sections which had been employed in the Batignolles tunnel after bending them to the profile of the Metro-

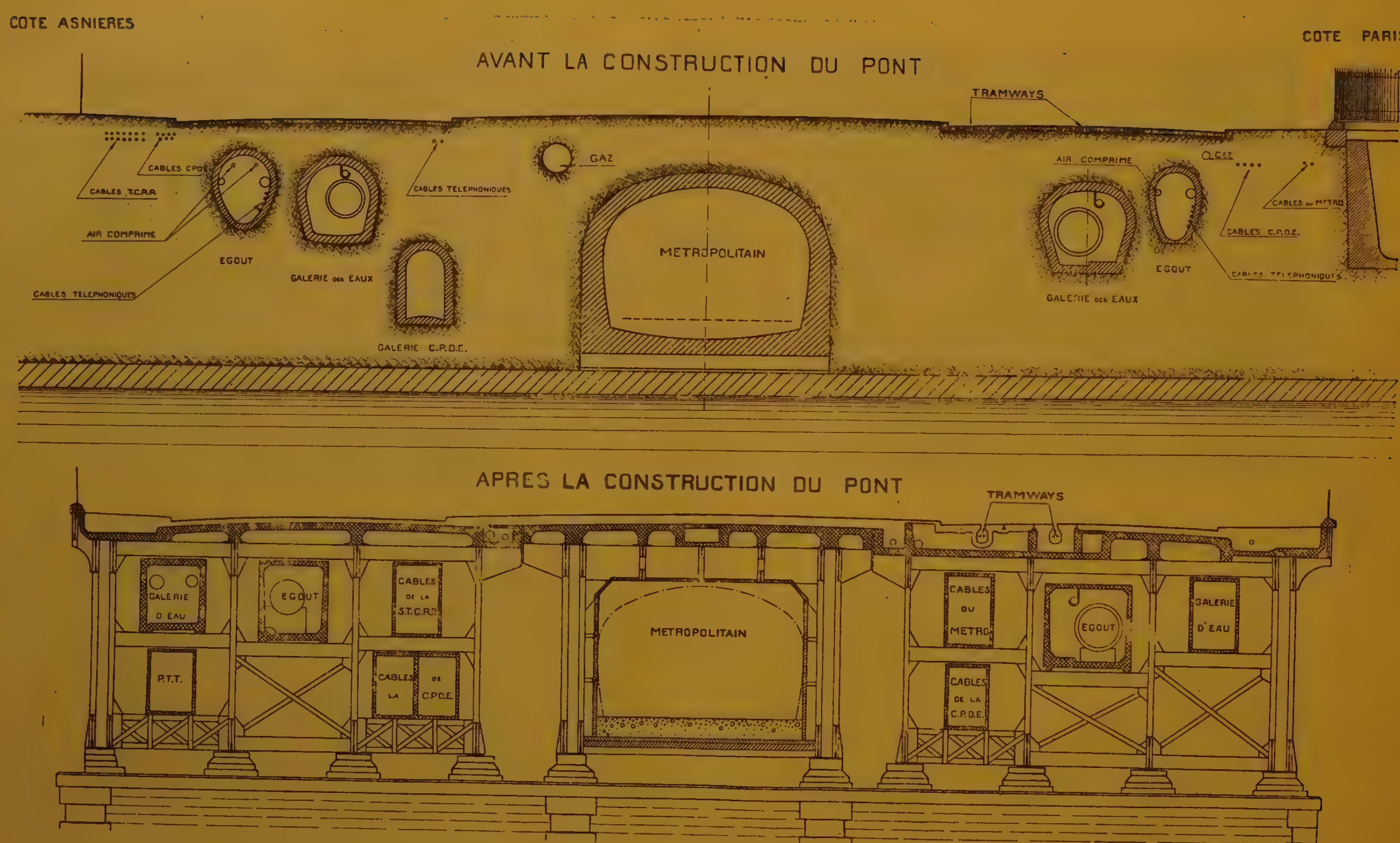



Fig. 19. — Boulevard des Batignolles bridge. — Ground before construction. — Cross section of bridge.

*Explanation of French terms* : Air comprimé = Compressed air. — Avant la construction du pont = Before construction of bridge. — Après la construction du pont = After construction of bridge. — Càbles téléphoniques = Telephone cables. — Côté Asnières = Asnières side. — Côté Paris = Paris side. — Egout = Sewer. — Galerie des eaux = Water mains. — Galerie C. P. D. E. = C. P. D. E. gallery. — Gaz = Gas

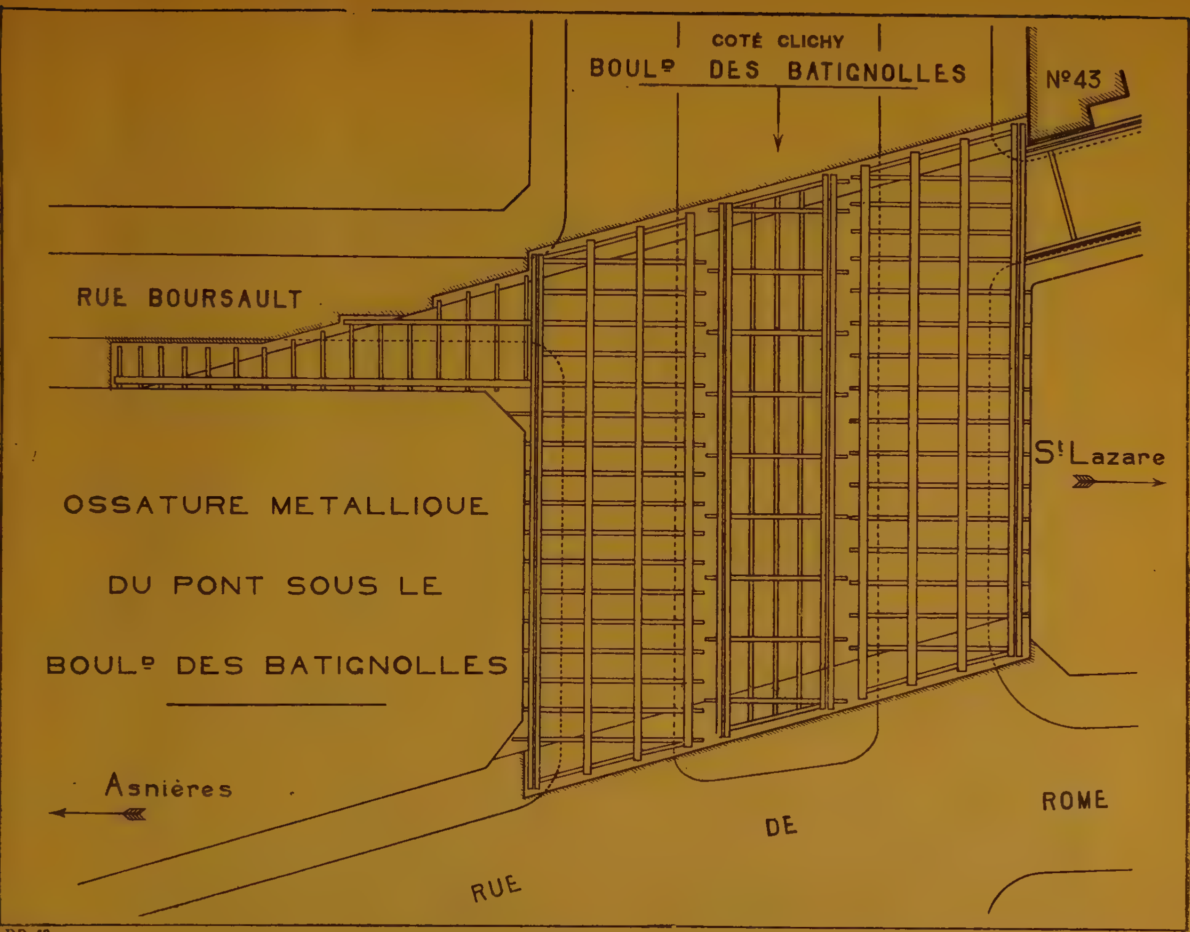



Fig. 20. — Boulevard des Batignolles bridge. — Plan view of framework.

*Explanation of French terms* : Côté Clichy = Clichy side. — Ossature métallique du pont sous le Boul[d] des Batignolles = Steel work of bridge carrying Boulevard des Batignolles.


Fig. 21. — Boulevard des Batignolles bridge.
View of works at commencement of construction of bridge carrying the Metro.

politain tunnel; the lining was then demolished.

At this time the works appeared as shewn by figure 21. In the distance is shewn a temporary wall (*a*) carrying the road on the Paris side, on which will be seen a tram proceeding towards Asnières, and at (*b*) a Metropolitain train can be seen running in the open. The centerings seen at (*c*) are some of those used when opening up the Metropolitain tunnel which had not been removed. The end of the tunnel and parts of the side wall not yet demolished may be seen at (*d*). The large crane in the foreground was used to erect the bridge carrying the Metropolitain; at (*e*) may be seen the first section erected of this bridge, and at (*f*) the commencement of the temporary shelter made of fire-proofed wood to protect the Metropolitain trains from falling tools or rivets.

The blocks of masonry (*g*) are narrow sections of the old invert of the Metropolitain tunnel intentionally allowed to remain; through the spaces between them the lower floor members of the bridge were inserted. The smallness of these openings was not the least of the difficulties of the work. Above these spaces, and between each block, temporary bearers to carry the rails of the Metropolitain were of course placed.

This is shewn more exactly by the working drawing given in figure 22. The rails of the Metropolitain are shewn at (*a*), the temporary bearers 5.20 m. (17 ft. 3/4 in.) long carrying them at (*b*), the narrow sections of masonry which carry the bearers at (*c*). The figure clearly shews that these sections were only remaining portions of the invert which was itself supported by concrete extending down to the linings of the railway tunnels, which are shewn at (*d*).

It was considered necessary, when the Metropolitain was built, to strengthen the linings with an additional course of masonry (*f*).

A real difficulty, after the sleepers and ballast of the Metropolitain were taken up and replaced by the bearers (*b*), was to make a way from below so as to be able to break into the very hard invert by cutting openings, which had to be spaced very accurately, for if they were too narrow, it would not have been possible to introduce the floor members (*g*), and if too wide, the narrow blocks under the Metropolitain might have been insufficiently wide, which would have been unsafe as regards the Metropolitain. A further difficulty was to place the floor members in their exact position, and do all this under the continuous and heavy traffic of the Metropolitain.

The jack arches carried by the floor members are shewn at (*h*).

This method of construction was the only one possible under the circumstances, and explains why the floor, instead of being formed, as is often the case, of main transverse members placed some distance apart and secondary longitudinal members, is in this case built up entirely of cross girders placed relatively close together, 1.30 m. (4 ft. 3 3/16 in.) centre to centre.

The construction of the rest of the bridge has now to be considered, as so far only the lower floor has been dealt with.

Figures 23 (a section across the Metropolitain) shews the corresponding operations.

One of the floor members just mentioned (*a*) is half hidden by one of the narrow masonry blocks, the lower boom (*b*) was erected, then the uprights (*c*), the upper boom (*d*) and the heavy floor members under the roadway (*e*) and, finally, the upper reinforced concrete under the roadway.

Especial care was taken when heavy details of the bridge members were being

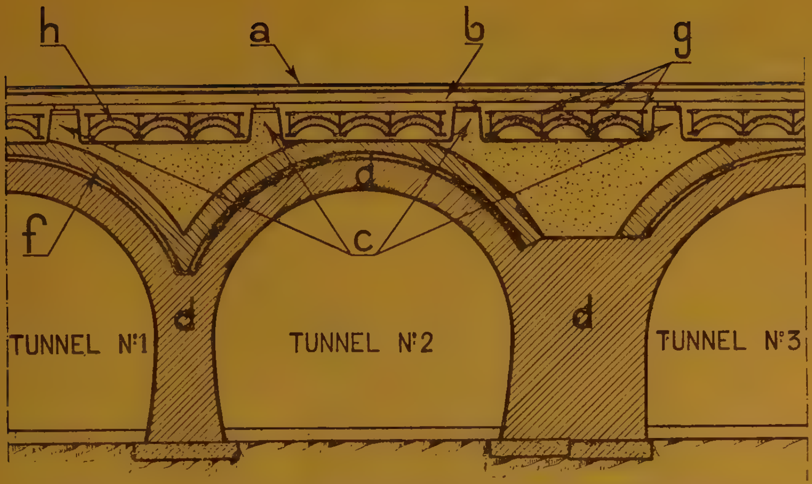



Fig. 22. — Construction of the lower flooring carrying Metropolitain line

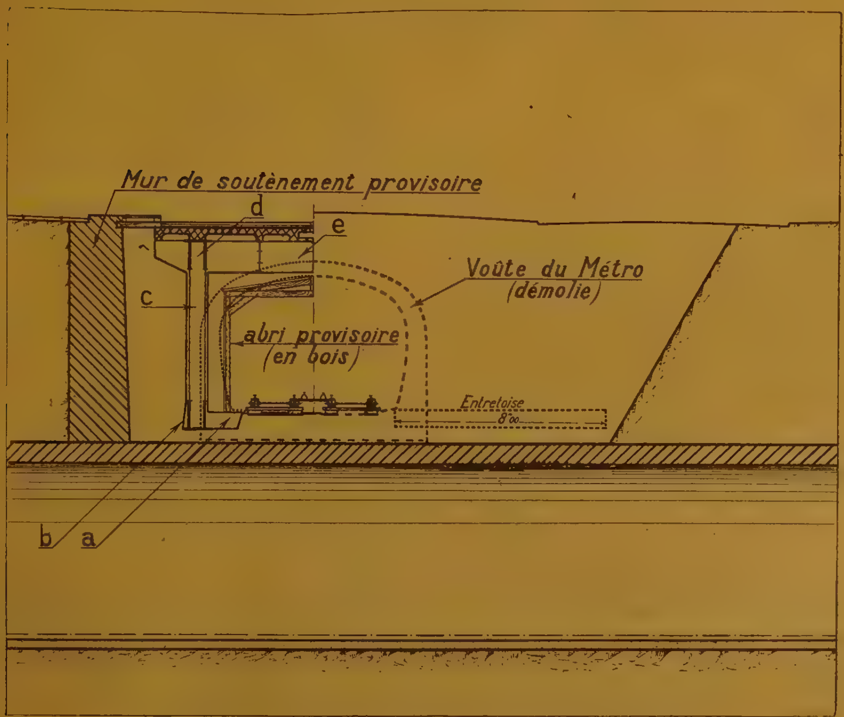



Fig. 23. — Construction of bridge carrying the Metro.

*Explanation of French terms* : Abri provisoire (en bois) = Temporary wooden shelter. — Entretoise = Floor member. — Mur de soutènement provisoire = Temporary retaining wall. — Voûte du Métro (démolie) = Metro tunnel demolished.

handled to ensure the safety, not only of the Metropolitain, but also of the public, whilst parts being taken from the dump by the cranes and lowered into their proper position were swinging from the end of the jib.

The description of the method of building the Metropolitain bridge and the various stages of the whole work will now be continued.

In order to slide the floor members 8 m. (26 ft. 3 in.) in length into their narrow spaces, they had to be arranged on the ground, as shewn in figure 23, ready for putting in position, that is to say, pointing in the correct direction. A space 8 m. (26 ft. 3 in.) wide parallel to the bridge had to be available. If the two side bridges had been first constructed, this space would not have been available. The Metropolitain bridge could not consequently be the third to be built.

It was, as a matter of fact, built first, and without entering into the various reasons which prevented it from being the second, it will be sufficient to give one of them. The town authorities had agreed that for a certain time the traffic might be carried on one of the two side roadways, but it quite reasonably required that this single roadway should not carry the tramway at the same time. By building the central Metropolitain bridge first, this condition could be easily met, because this bridge was ready to carry the deviated tramway by the time the construction of the bridge on the Asnières side had only left the single roadway on the Paris side in use.

Some details on the various stages of the work are given below.

Figure 24 shews the more important preparatory operations. The section shews at (*a*) two temporary retaining walls which were necessary to prevent, on the left road on the Paris side, on the right the previously relocated mains on the Asnières side, from falling into the excavation of the Metropolitain.

The left-hand wall happened to come just below the tramway, which was relayed over the Metropolitain, as is shewn on the plan, in order to construct the temporary wall, after which it was restored to its old place. (It will be seen later that it was replaced over the Metropolitain bridge as soon as built, for the reason already given, and finally it was moved a fourth time to its final position on the bridge on the side towards Paris.)

During this preparatory stage, the greater part of the abutments of the bridge (*b*) in figure 28 were constructed. Finally, the mains (*c* in the figure) were moved to the Asnières side in order to clear the ground for the excavations to be made in connection with the Metropolitain bridge, and later for the construction of the bridge towards the Asnières side. On the right hand of No. 62, Boulevard des Batignolles, (point *d* on the figure) a width of 2.50 m. (8 ft. 2 3/8 in.) was all that was available; the following mains :

Water :

1 main 1.100 m. (3 ft. 7 5/16 in.);
1 — 0.500 m. (1 ft. 7 11/16 in.);
1 — 0.400 m. (1 ft. 3 3/4 in.);

28 cables of the C. P. D. E.;
18 — of the T. C. R. P.;

1 compressed air main;
1 pneumatic tube;
1 sewer;

were successfully moved there by laying them either side by side or one above the other.

To arrange all these mains in so small a space required much good will from all.

Figure 25 shews the stages of uncovering the Metropolitain tunnel and its demolition.

Figure 26 shews the erection of the Metropolitain in its relative place.

During this period, as the figure shews, two roadways could still be left open to

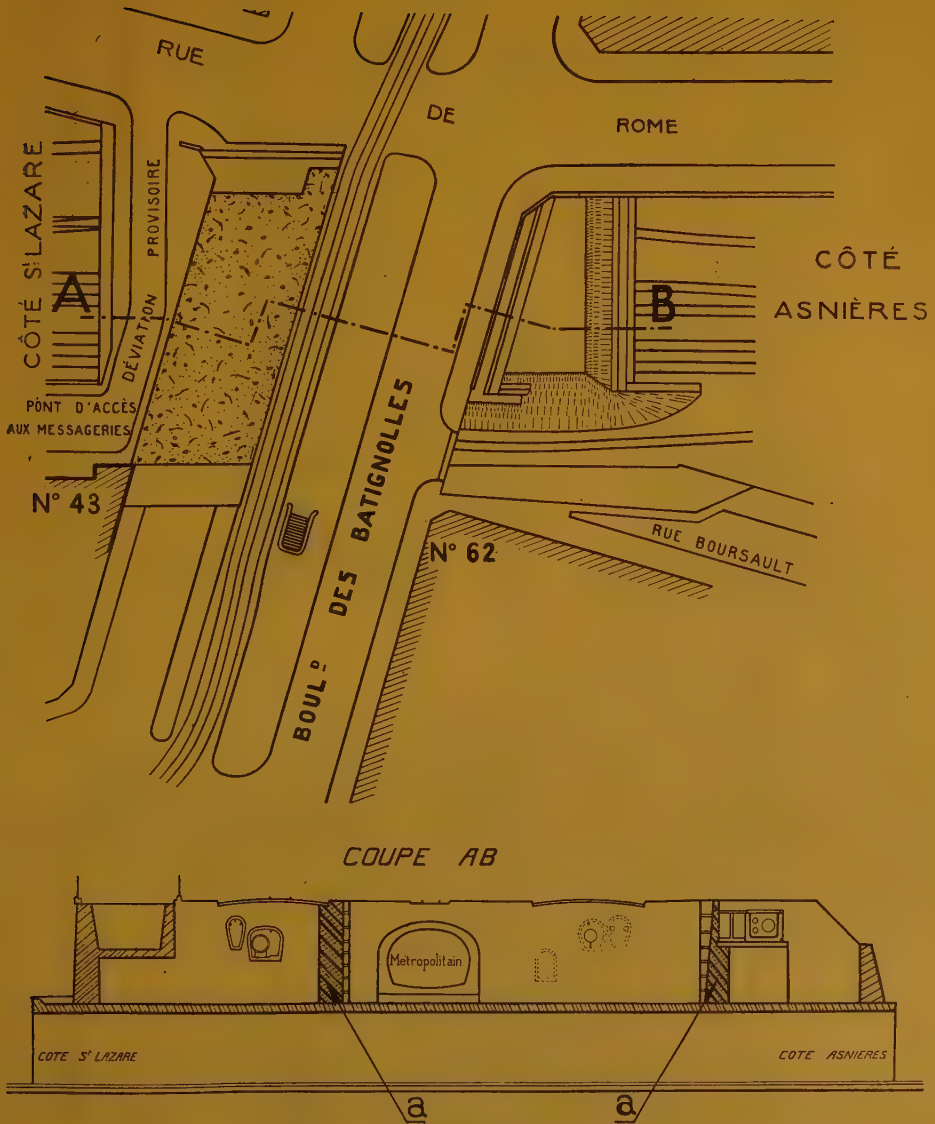



Fig. 24. — Construction of Batignolles bridge. — Preparatory work.

*Explanation of French terms* : Coupe AB = Section AB. — Déviation provisoire = Temporary deviation. Pont d'accès aux Messageries = Bridge giving access to the Messageries building.

the public; the right hand road is of course temporary.

Figure 27 shews the construction of the second bridge (side towards Asnières) with one of the two roadways gone, but with the tramway on the Metropolitain

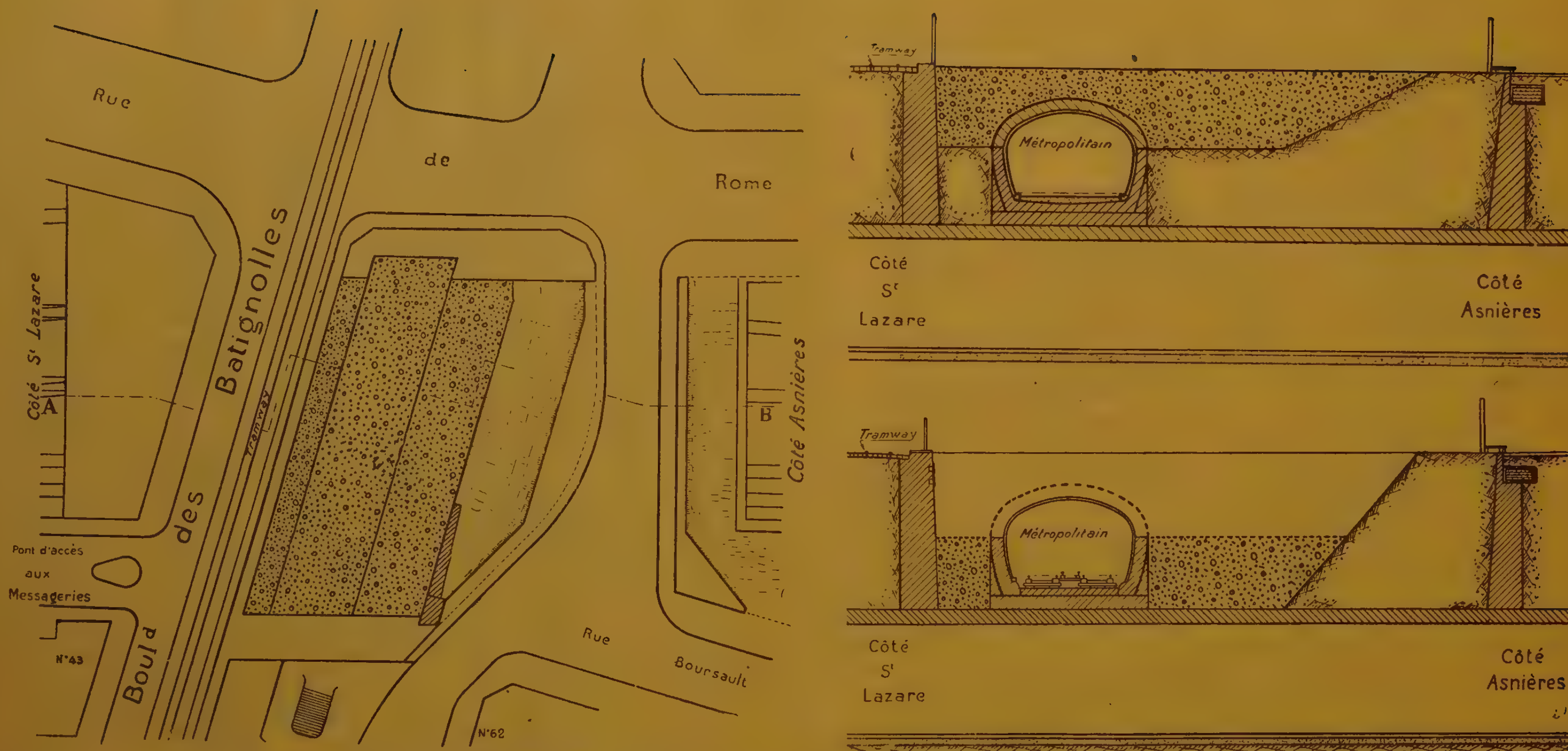



Fig. 25. — Construction of Batignolles bridge. — Demolition of Metro tunnel. — Excavations for the Metro bridge.

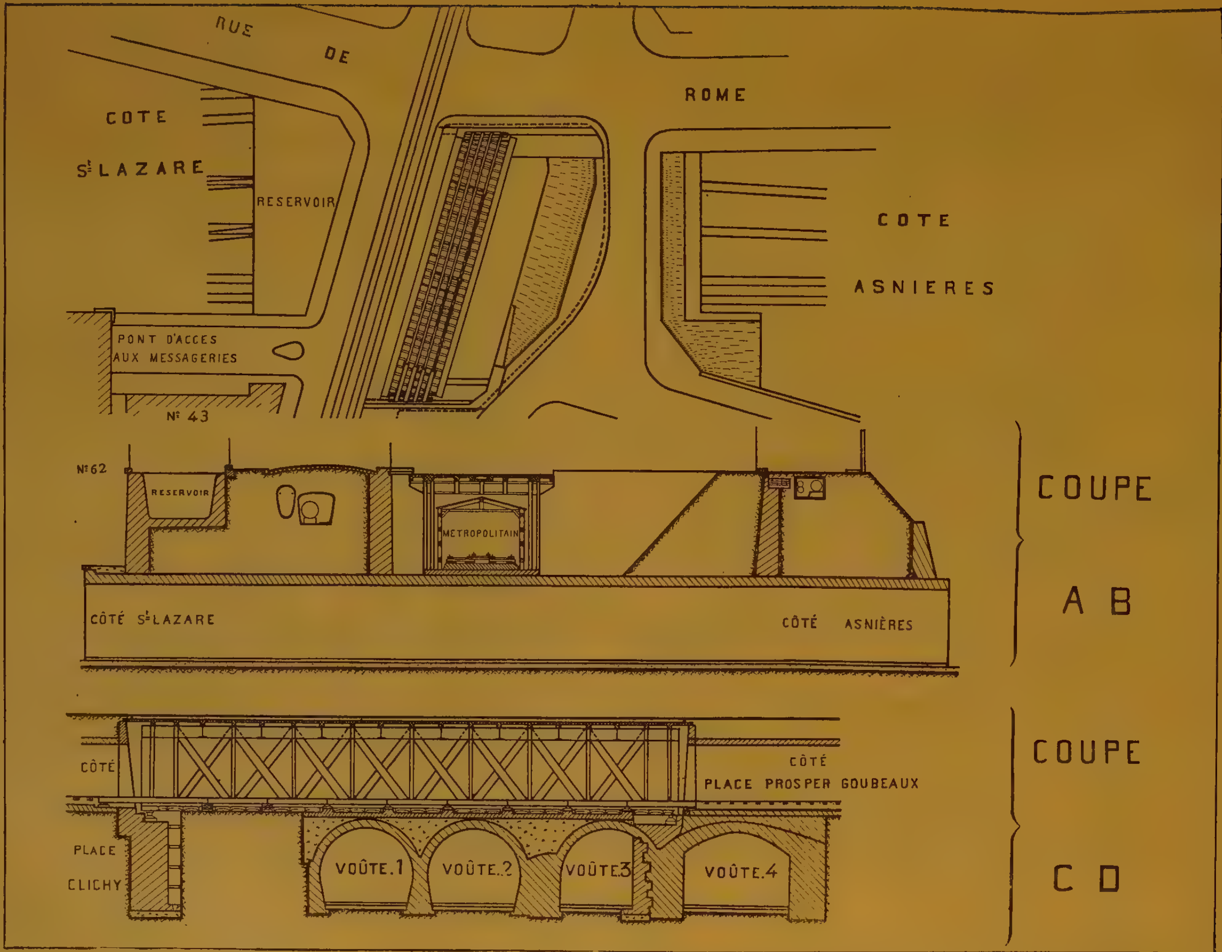



Fig. 26. — Construction of Batignolles bridge. — Plan of works during construction of Metro bridge.

*Explanation of French terms* : Coupe AB = Section AB. — Coupe CD = Section CD.

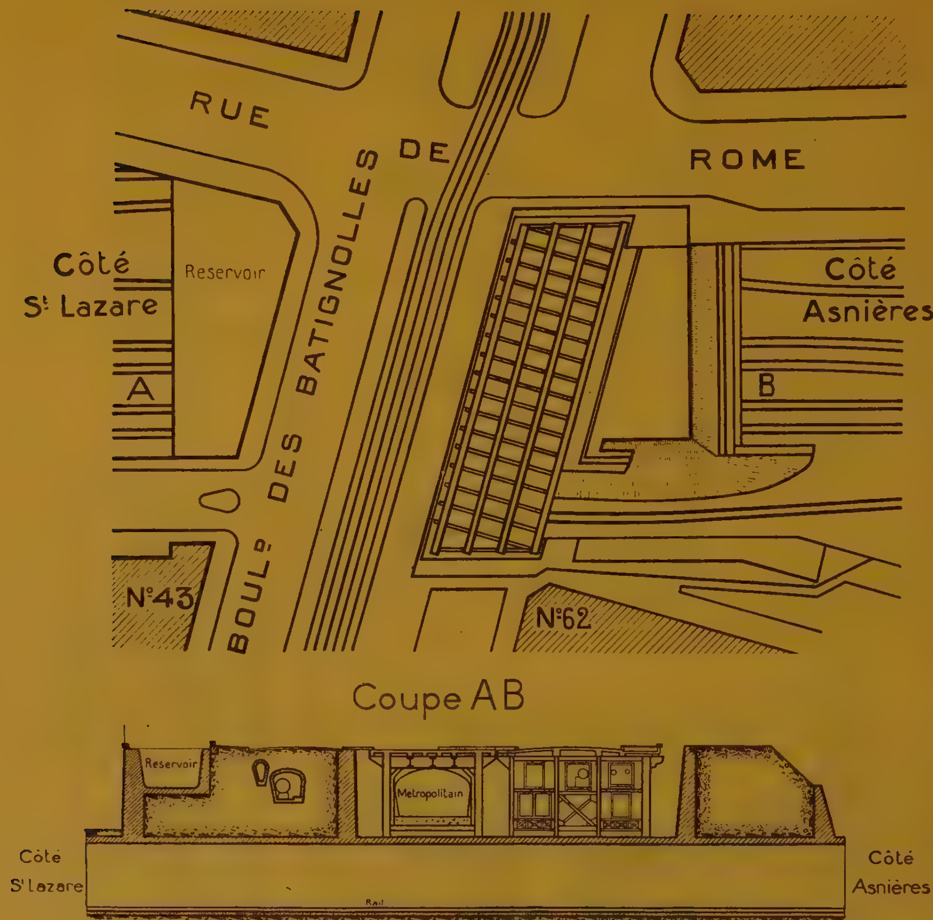



Fig. 27. — Batignolles bridge. — Construction of second bridge (Asnières side).

bridge. This deviation of the tramway itself necessitated fairly important strengthening of the roof of the booking office at the Rue de Rome station, and also of the steel roof over the lines and platforms at the station, which had not been designed to carry a double line of tramway. Four strengthening girders, 1 m. (3 ft. 3 3/8 in.) deep and 14 m. (45 ft. 11 in.) span, were put in place during the night, a difficult operation in view of the limited time available. Bringing up these heavy girders was fortunately facilitated by the contractor's workshops (Laurent-Moisant-Savey Works) being connected to the Metropolitain at Ivry, and by using service wagons belonging to the Metropolitain.

Figure 28 shews the excavation necessary for the third bridge on the side towards Paris. Traffic was now carried on the new permanent roadway on the Asnières side, but another temporary roadway had to be made on the Paris side (*a*) to give access to the Messageries building.

It must be understood that certain mains had to be relayed on the Paris side as on the Asnières side mentioned above. The number of mains was less on this side, but on the other hand, the work has been somewhat difficult, as owing to the existence of No. 43, Boulevard des Batignolles, they all had to pass through the cellars of the building and, after the cellars, through the steel foot-bridge giving access to the Messageries building.

At the present time this last stage is being completed, and the third bridge is nearly finished.

The progress of the work has followed the prearranged programme exactly, but was complicated by an important reason about which something should be said.

Towards the end of 1923, progress on the whole of the works at Paris, Cardinet, Clichy, Asnières, Bécon and Bois-Colombes, led us to believe that electrification of the first section could take place in about April of the year following, that is to say, 1924. If the extension of the auxiliary lines as far as Paris (one of the objects mentioned at the beginning of this article, which could be gained by the opening out of the tunnels) could wait, it would, on the other hand, be impossible to electrify, without having made the Saint-Germain and Argenteuil groups of lines independent of one another. However, the first and second portions of the work were almost finished, and the eight necessary roads were already available (it was in fact necessary to have eight roads for the four groups : Versailles, Saint-Germain, Asnières, Main Lines); but on the right of the boulevard the position would not allow this number of roads to be made available. The four tunnels only accommodated seven, owing to the space taken in the third tunnel by the strengthening of the side wall, frequently mentioned above. Figure 29 shews this more particularly (to the five roads through the tunnels shewn in this figure must of course be added the two roads in the remaining tunnel on the Rue de Rome side.)

The eighth road has, however, been completed in good time by working it in between No. 1 tunnel and the abutment of the bridge, as is shewn very clearly on the left hand of this figure, but certain quite special arrangements had to be made, and in particular it was necessary to erect the first section about 10 m. (33 feet) long (Clichy side) of the Metropolitain bridge quickly. This is shewn in figure 21.

Without describing the special measures taken at this time, it may be mentioned that the earlier than expected construction of the first section of the Metropolitain bridge gave the ground an unusual appearance. A temporary abutment was put in to carry the end of this section of the bridge for the time being. At this time the excavation to the right of the Metropolitain bridge had not been made. The excavation for the two abutments (one permanent and the other temporary) for this section of the bridge, each necessitating cutting through the roof of the Metropolitain tunnel, had therefore to be carried out with a crane situated above the Metropolitain working between them. The excavations had to be timbered with very great care. When the two excavations were made and the excavating machinery removed, an isolated section of the roof of the tunnel was left between them, and was later demolished under rather special conditions.

Any delay in the electrification of the first section (Paris-Bécon-Bois-Colombes), which took place on the 21 April 1924, was successfully avoided.

During the execution of these works, a number of problems arose, of which it may perhaps be of interest to mention the under-pinning of No. 43, Boulevard des Batignolles. This building, which is

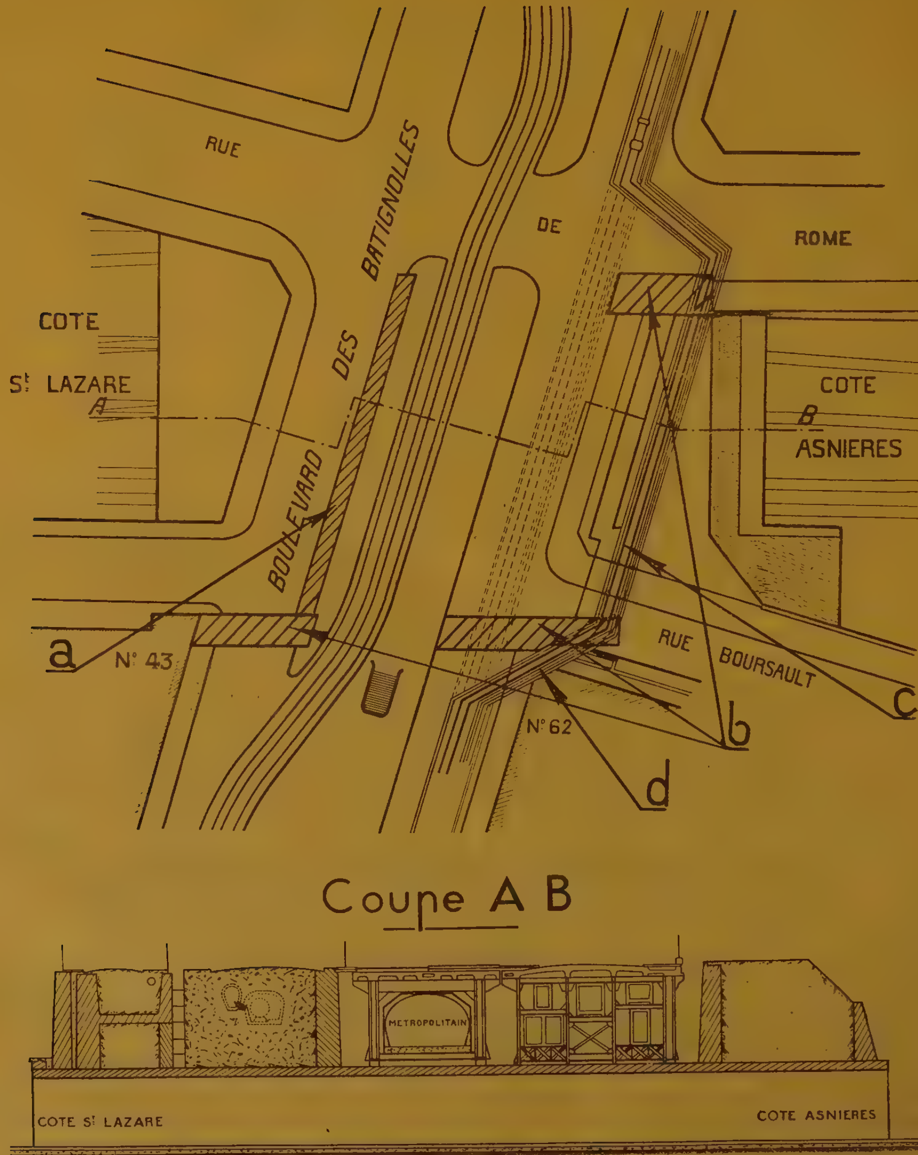



Fig. 28. — Batignolles bridge.

Excavation for the third bridge (Paris side).

shewn in figure 28, had shallow foundations and the eighth road, mentioned above, whether constructed before or after the completion of the bridge, would of necessity run along the front of this building, but at 17 m. (55 ft. 9 1/4 in.) below the ground floor. As the foundations were shallow, it had to be under-pinned to this depth of 17 m. which was not easy. The adjoining works made it awkward, and it was difficult to shore up the building, on the boulevard side, because of the narrow pavement and the adjacent excavation, and on the railway side owing to the Messageries foot-bridge through which holes had to be cut to gain access to the solid ground (itself pierced by a short temporary tunnel for a line which had to be made).

As an appendix is given a note which M. Le Pécheur, Assistant Divisional Engineer of the State Railways, kindly prepared on this portion of the work, the interest of which lies in the number of local restrictions which accompanied under-pinning to such an unusual depth.

Before concluding, the perfect spirit of co-operation, which throughout existed between the various public departments affected by the work in connection with the tunnel, should be mentioned.

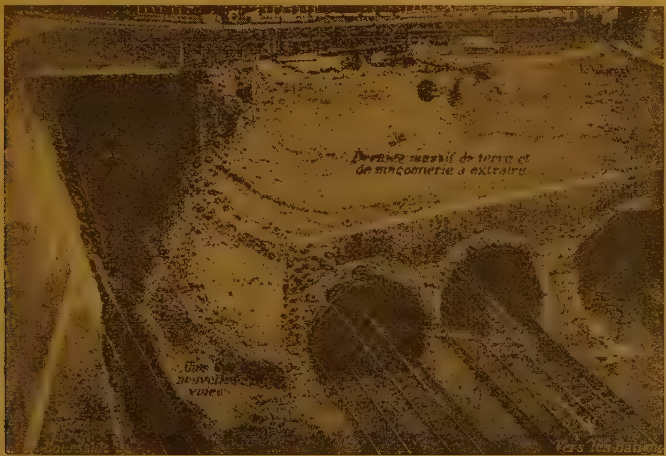


Fig. 29. — Boulevard des Batignolles bridge.
Provision of the eighth road necessary for electrification.

*Explanation of French terms* : Dernier massif de terre et de maçonnerie à extraire = Last block of the earth and masonry to be removed. — Anciens tunnels = Old tunnels. — Une des nouvelles voies = One of the new lines. — Mur soutenant la rue Boursault = Retaining wall, rue Boursault.

It is due to their co-operation and to the capability of the contractors (Messrs. J. et J. Combe, for the excavations, masonry and demolition of the whole of the tunnels, and the Laurent-Moisant-Savey Works for the construction of the Boulevard des Batignolles bridge), to the opportunities which we had in scheming out the various details of the work and to the constant attention given to its execution, that up to the present the whole of the demolition works at the Batignolles tunnels has been carried out without any accidents.

When the bridge was nearly finished, the demolition of the last portion of the tunnels (30 m.) (98 ft. 5 1/8 in.), which is situated under the bridge was com-

menced, and the linings have already been demolished for nearly the whole of this length. This excavation and masonry from beneath the bridge were brought by narrow gauge track immediately under the outside girder on the Asnières side, and the hoppers were there raised by an electric hoist running on a monorail and taken to the right-hand footpath of the Rue de Rome and emptied into a hopper, from which the spoil was loated into lorries. This mono-rail itself was fixed on supports carried on the footpath on the bridge. This plant is now in full operation.

It is hoped to remove the centering, that is to say, to completely finish the whole of the work, in a few months.

---

## SUPPLEMENTARY NOTE on the under-pinning of No. 43, Boulevard des Batignolles.

---

The third stage of the work of opening out the Batignolles tunnel (construction of a bridge under the Boulevard des Batignolles) included among the « Preparatory work », *the under-pinning and strengthening of the foundations of No. 43, Boulevard des Batignolles.*

This building consists of two parts : the first MNOP, figures 31 and 34, facing on the boulevard with a side on the ground alongside the railway; the second part facing the railway along PQR.

As the ground is particularly good, the foundations of this first part originally were of moderate depth only, except under the front wall, which was built on ground previously excavated when building a boundary wall where it was for this reason supported at 1.80 m. (5 ft. 10 7/8 in.) below the ground level, on a masonry footing, about 1 m. (3 ft. 3 3/8 in.) thick, and 4 m. ( 13 ft. 1 1/2 in.) wide (see fig. 32).

On the railway side the foundations originally extended 0.85 m. (2 ft. 9 1/2 in.) below the level of the cellar. In 1917 they were carried down a further 2.75 m. (9 ft. 5/16 in.) with an average thickness of only 0.70 m. (2 ft. 3 9/16 in.) when the Messageries bridge was built.

A retaining wall 13 m. (42 ft. 8 in.) high above the formation level at Saint-Lazare station was used as the foundation on the railway side for the second part of the building. On the slope between P and Q the foundations of this retaining wall did not extend down to rail level before the strengthening work was carried out. They were stepped, as shewn in figures 30 and 36, and only stopped when solid ground was reached.

The general appearance of this building before the under-pinning and strengthening of the foundations is shewn in the elevation (fig. 30).

The side wall NO standing approximately in line with the prolongation of the outer face of the abutment of the Batignolles bridge, it was necessary, not only to carry the foundations down to the formation level of Saint-Lazare station, but also to construct under the building a real retaining wall, capable of supporting any loading which might come on it, in this very special case.

The construction of this retaining wall under a five storey house with attics, completely occupied was in the circumstances a delicate matter.

MNOP was the first part to be dealt with. Before commencing excavation, and in order to avoid any movement during the under-pinning, the building was shored up, as shewn in figures 31 to 33. For this purpose, after bracing the portions of the front and side walls concerned, groups of shoreing timbers were placed as follows : 2 on the front facing the boulevard, 5 on the side facing the railway, and 1 at the corner between the front and side wall, as shewn in figure 31.

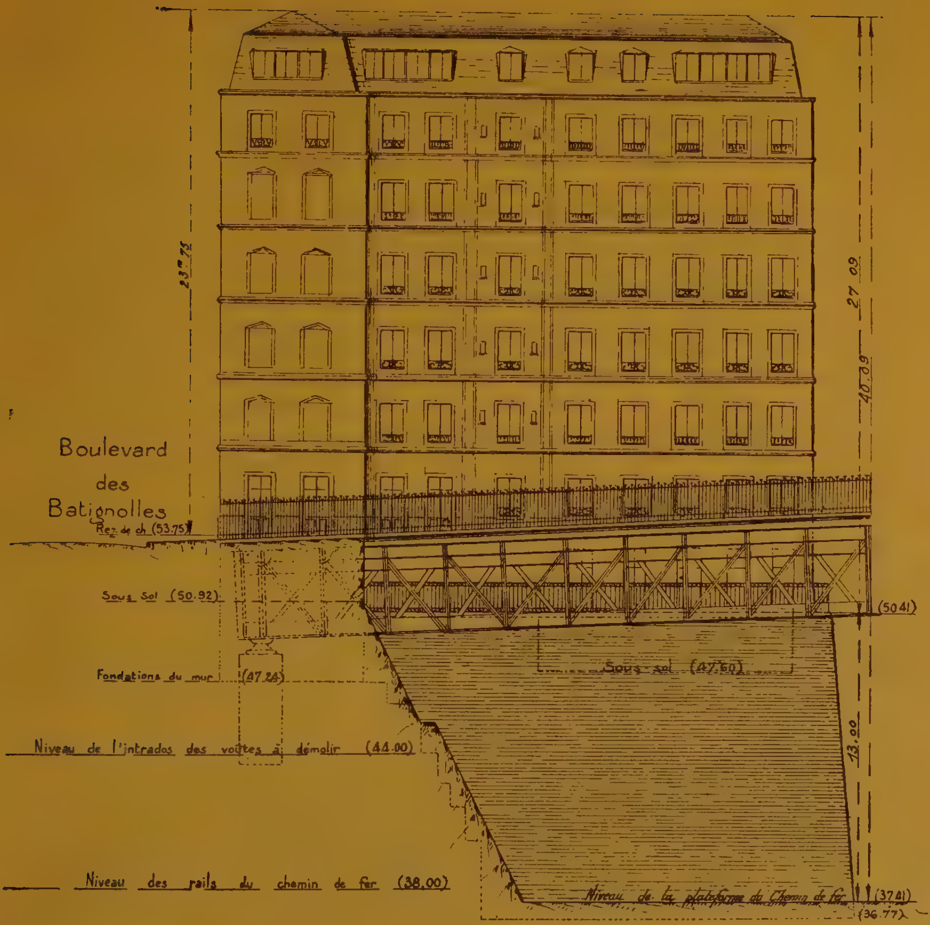



Fig. 30. — No. 43 Boulevard des Batignolles. — General view of the building from the railway.

***Explanation of French terms of figs. 30 and 31*** : Appuis provisoires du pont d'accès aux Messageries = Temporary abutments for Messageries bridge. — Axe de la poutre de rive du pont des Messageries = Centre line of outer girder of Messageries bridge. — Axe du tunnel provisoire = Centre line of temporary tunnel. — Bois d'étaiement = Struts. — Fondations du mur = Foundation of wall. — Fouille pour culée paire du pont des Batignolles = Excavation for abutment of Batignolles bridge. — Limite du tablier du pont, etc. = Outer edge of roadway, Messageries bridge. — Niveau de la plateforme du chemin de fer = Railway formation level. — Niveau de l'intrados des voûtes à démolir = Level of crown of tunnels to be demolished. — Niveau des rails du chemin de fer = Rail level. — Parement de la culée = Inner edge of abutment. — Pont provisoire sous chaussée = Temporary bridge carrying road. — Rez-de ch. = Ground floor. — Sommier métallique destiné à supporter le pont d'accès aux Messageries = Steel abutment to carry Messageries bridge during construction of temporary tunnel. — Sous-linteau (7 fers I de 280) = Support (seven 280 mm. (11 inch) I girders). — Sous-sol = Basement. — Voie Decauville = Narrow gauge track. — Voie de travaux = Temporary track.

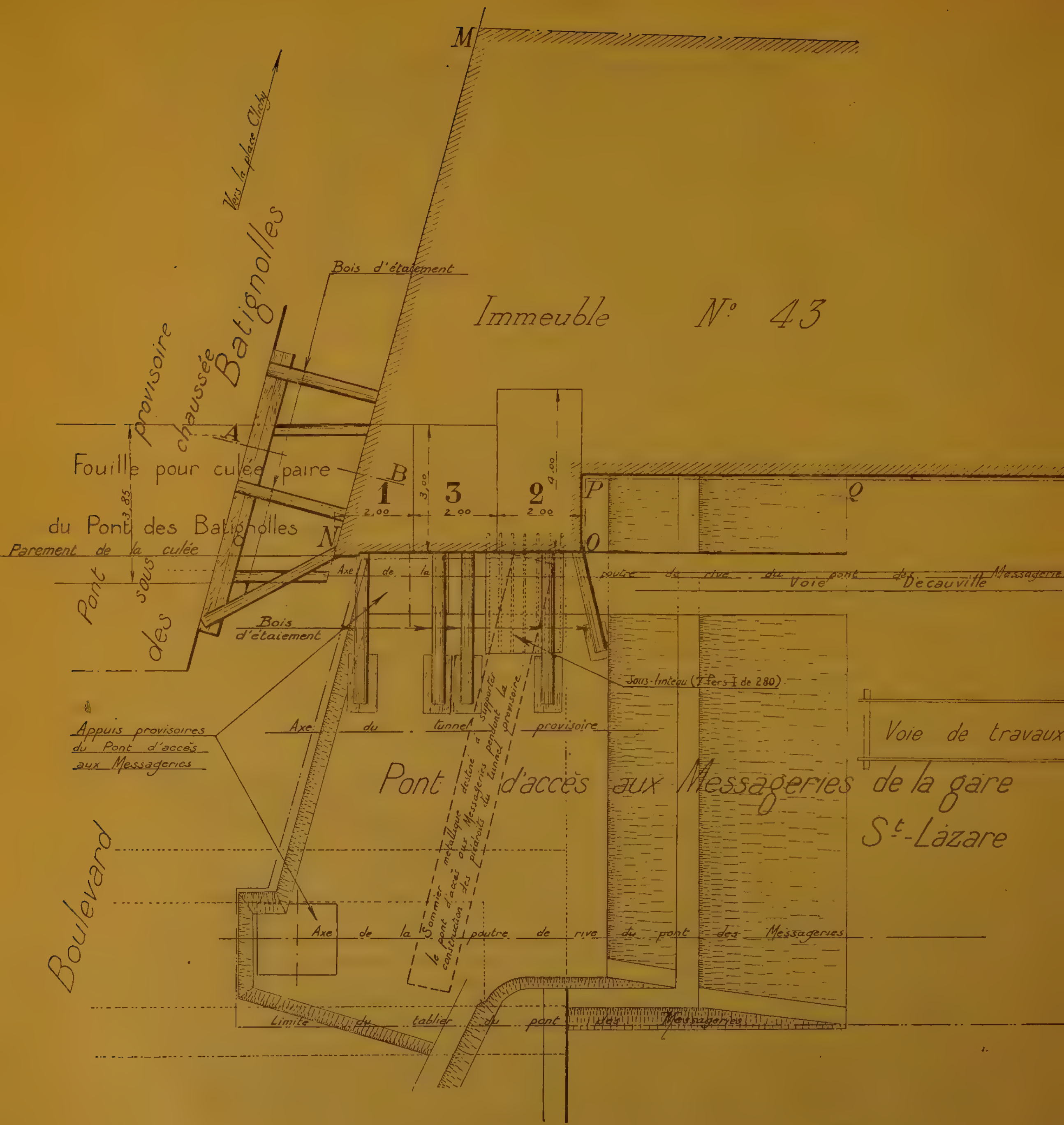



Fig. 31. — Shoreing of No. 43 Boulevard des Batignolles.

All these shores were formed of oak logs about 0.35 m. (13 3/4 inches) diameter. Those placed at the front and at the angle of the building were supported on two baulks 0.30 m.×0.30 m. (11 13/16 inches × 11 13/16 inches) square laid across the 3.85 m. (12 ft. 7 1/2 in.) trench opened out when building the abutment of the bridge under the boulevard (see fig. 32); those of the side wall passed through the flooring of the Messageries bridge at BA and rested on the solid ground (see fig. 33).

Heavy girders each built up of several rolled steel joists, with one end in the cellar of the building and the other end on the solid ground under the Messageries bridge, were passed through the side and front walls to support the whole house during the excavation and subsequent concreting under the building.

As shewn in figures 34 and 35, this operation was very difficult owing to the length of these girders, the shoreing timbers already in place and the existing steel abutment put in to carry the Messageries bridge during the construction of the side walls of the temporary 5.70 m. (18 ft. 8 7/16 in.) tunnel. The concrete block alongside No. 43 of the boulevard, provisionally carrying the Messageries bridge, was under-pinned to form one of the side walls of the temporary tunnel, and the block on the other side, being at a tangent to the other side wall, it was necessary to relieve them of all pressure while work was going on immediately around them, and it was for this reason that the steel abutment above mentioned was used [1].

The work of shoreing up and securing the building took a month, from the 6 February to the 6 March 1923. When this was completed, shafts were sunk, each being about 2 m. (6 ft. 6 3/4 in.) wide. Shafts Nos. 1 and 2 were sunk simultaneously, and were only separated by 2 m. (6 ft. 6 3/4 in.), which was however sufficient in view of the nature of the earth (compact marl and soft limestone).

The soil excavated from shaft No. 1 was raised by crane and transported by lorries to public dumps at the same time as that taken from the excavation for the abutment of the bridge carrying the boulevard. That taken from shaft No. 2 was removed to Saint-Lazare station by a 1.50 m. (4 ft. 11 in.) heading made along the site of the side wall of the temporary tunnel, the level being lowered to suit the shafts. The soil was then loaded into wagons and taken by ballast trains to dumps on the railway.

The excavation of these two shafts was commenced on the 7 May 1923, and finished on the 20 April following. The concreting of shaft No. 2 was carried out between the 20 and 26 April, and of shaft No. 1 between the 24 April and 4 May.

During this first part of the work, the Messageries bridge was carried on its two temporary abutments, so that it was possible to excavate without difficulty under the end of the steel abutment and build the heading for removing the earth from shaft No. 2. Shaft No. 3 could not be taken in hand without lengthening the heading up to the temporary concrete abutment of the Messageries bridge, and the said concrete had to be partially under-pinned during the sinking of the shafts. In order to form the side wall of the temporary tunnel, it was necessary to relieve the weight on the temporary abutments of the bridge and to transfer the load to steel abutment previously put in place for this purpose.

To do this, it was first of all necessary to support the overhanging steel abutment. This is the reason for the use

[1] When completed the Messageries bridge will be supported by the outer guider of the Boulevard des Batignolles bridge.

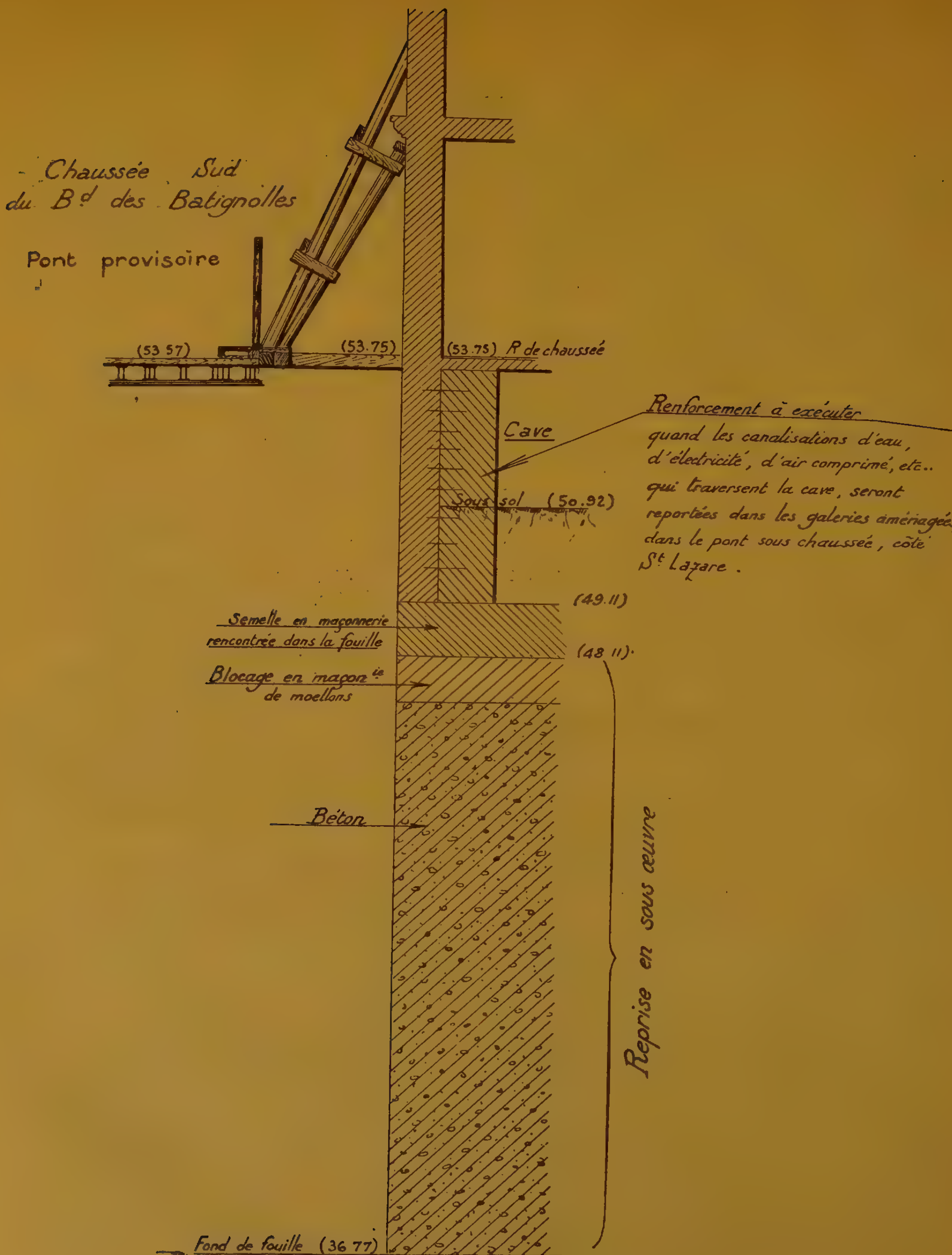



Fig. 32. — Shoreing of No. 43 Boulevard des Batignolles.
Section along AB on figure 31.

*Explanation of French terms*: Béton = Concrete. — Blocage en maçonnerie de moellons = Rubble filling. — Cave = Cellar. — Chaussée Sud, etc. = South roadway of Boulevard des Batignolles. — Fond de fouille = Lowest point of excavation. — Pont provisoire = Temporary bridge. — Renforcement à exécuter quand, etc. = Strengthening to be carried out when the water electric and compressed air mains running through the cellar have been transferred to the galleries in the bridge under the road on the Saint-Lazare side. — Reprise en sous-œuvre = Under-pinning. — Semelle en maçonnerie, etc. = Masonry footings found during excavation.

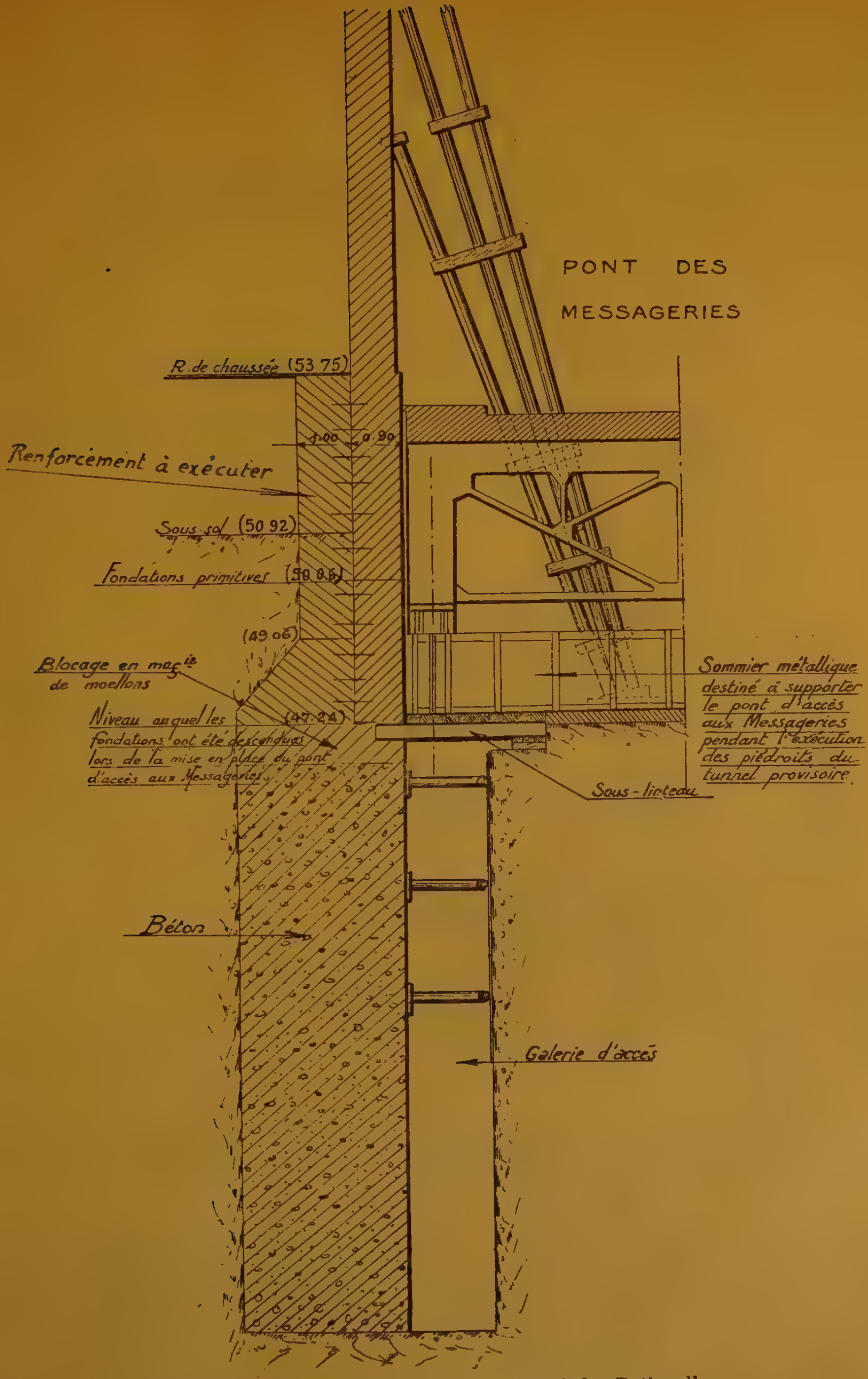



Fig. 33. — Shoreing of No. 43 Boulevard des Batignolles.
Section showing the struts placed on the Messageries bridge side.

*Explanation of French terms* : Béton = Concrete. — Blocage en maçonnerie de moellons = Rubble filling. — Fondations primitives = Original foundations. — Galerie d'accès = Approach heading. — Niveau auquel les fondations ont, etc... = Level to which the foundations were taken down at time of construction of Messageries bridge. — Pont des Messageries = Messageries bridge. — Renforcement à exécuter = Strengthening to be carried out. — R. de chaussée = Ground floor level. — Sommier métallique destiné à etc..., = Steel abutment to carry Messageries bridge during construction of the side walls of the temporary tunnel. — Sous-linteau = Support. — Sous-sol = Basement.

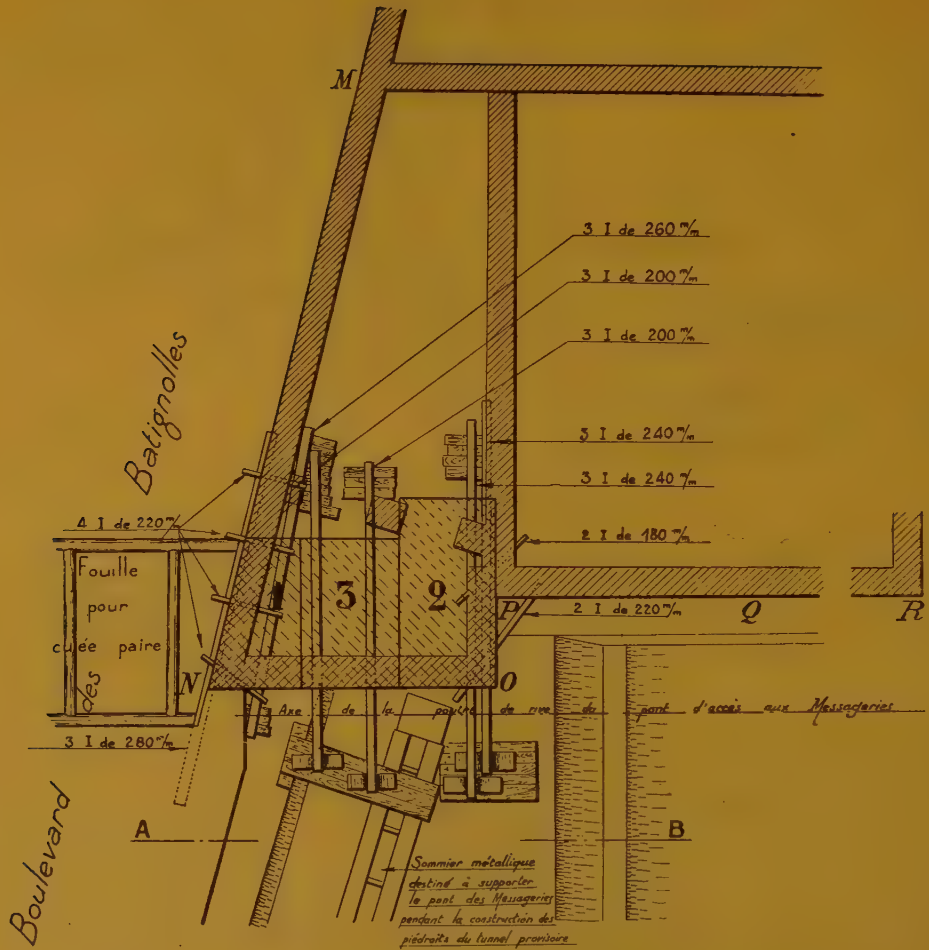



Fig. 34. — Supporting No. 43, Boulevard des Batignolles, by means of girders.

*Explanation of French terms* : Axe de la poutre de rive, etc. = Centre line of outer girder of Messageries bridge. — Fouille pour culée paire = Excavation for abutment of bridge. — Sommier métallique, etc. = Steel abutment to carry Messageries bridge during construction of temporary tunnel.

of the support, which will be seen in figure 33. This support was formed of seven 280 mm. (11 1/32 inch) joists, one end built into the newly constructed wall (shaft No. 2) and the other end resting on the ground.

When this was done, shaft No. 3, extending half the length of the temporary abutment, was excavated and concreted without any difficulty between the 2 June and the 23 July, using the heading which had already been used for shaft No. 2 for removing excavated soil and bringing up material.

The whole of the retaining wall, which was thus made in three stages, was con-

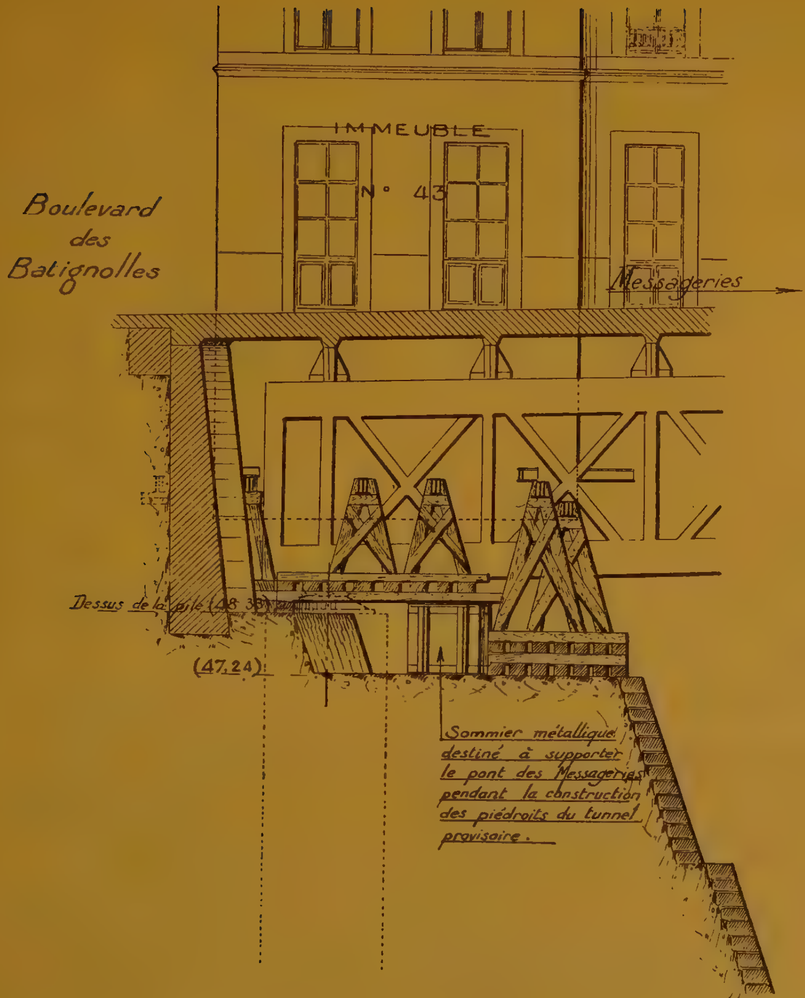



Fig. 35. — Supporting No. 43, Boulevard des Batignolles, by means of girders. Section through A on figure 34.

structed of concrete formed of gravel and slag cement; but where joined up with the old foundations of the building, it was built for a height of 0.60 to 0.80 m. (1 ft. 11 5/8 in. to 2 ft. 7 1/2 in.) of hard rubble in Portland cement mortar, because this latter form of masonry facilitated the construction of a solid wall without any empty spaces. In addition to this precaution, injections of Portland cement were made after the completion of the wall at the joint between the new and old masonry to compensate for the shrinkage of the concrete.

The supports and shores were removed,

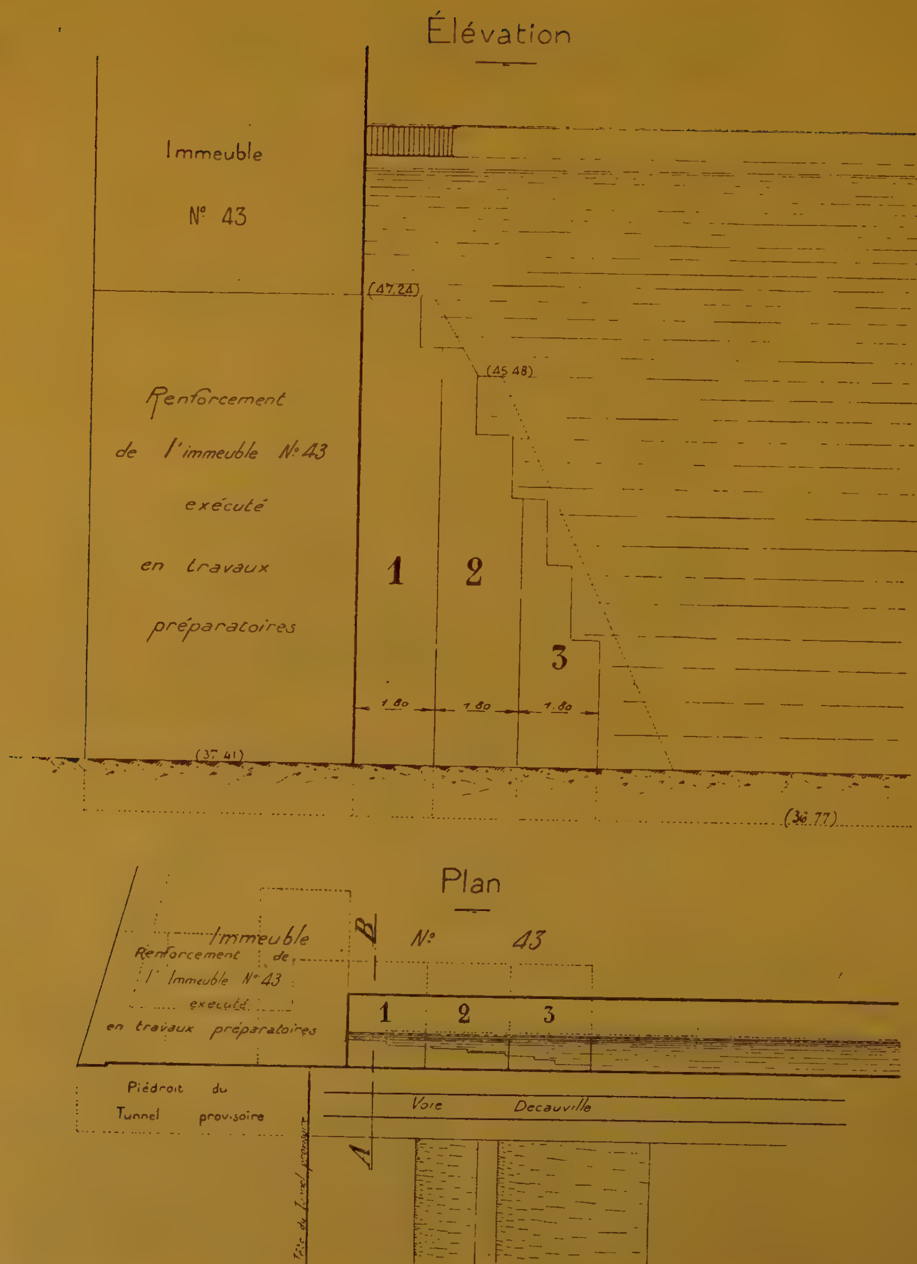



Fig. 36. — Under-pinning of foundations of No. 43, Boulevard des Batignolles. — Plan and elevation of retaining wall along side of railway.

*Explanation of French terms* : Piédroit du tunnel provisoire = Side of temporary tunnel. — Renforcement de l'immeuble n° 43, etc. = Under-pinning of No. 43 carried out as preparatory work. — Tête du tunnel provisoire — End of temporary tunnel. — Voie Decauville = Narrow gauge track.

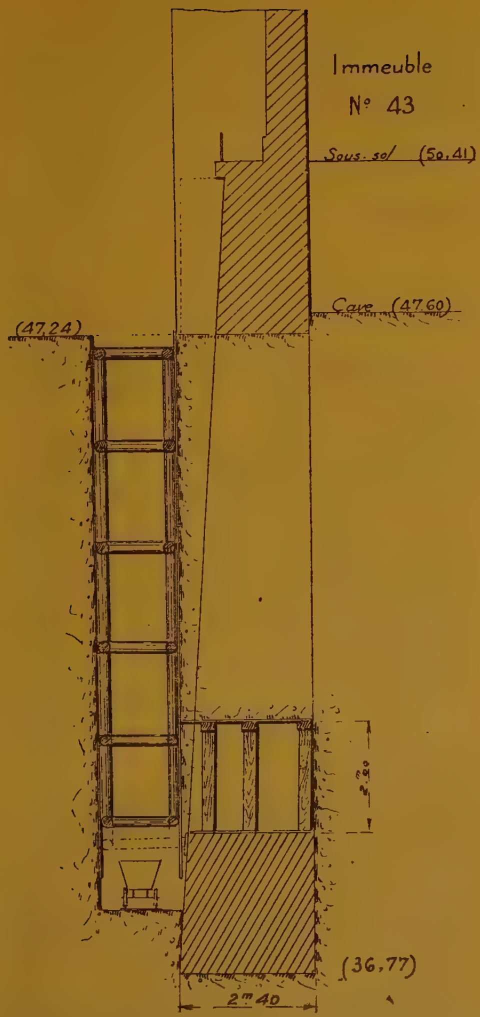



Fig. 37. — Under-pinning of foundations of No. 43, Boulevard des Batignolles. — Section through AB on figure 36.

*Explanation of French terms* : Cave = Cellar. Sous-sol = Basement.

after these injections, from the 1 to 5 August 1923. No movement of the walls of the building took place either during the work or since its completion two and a half years ago.

When this work was finished, the under-pinning of the part of the retaining wall along the side of the railway between P and Q in figures 31 and 34 was still to be done : it was commenced on the 31 August 1923. Figures 36 and 37 shew the extent of the under-pinning which had to be carried out, and the method adopted for doing it.

The area free from foundation had been previously cleared during the construction of the heading for evacuating the soil from shafts 2 and 3 of the work finished in the early days of August. This area was divided in elevation into three cuts, each about 1.80 m. (5 ft. 10 7/8 in.) long. In each of these the wall was under-pinned, starting from the base, by stages about 2 m. (6 ft. 6 3/4 in.) high, as shewn in figure 37. Material was removed and brought up by the heading already used for the same purposes previously.

The method adopted allowed the work to be carried on quite safely, since it was only necessary for each stage to excavate under the wall a heading 1.80 m. (5 ft. 10 7/8 in.) wide and of a length varying from 2.40 m. (7 ft. 10 7/16 in.) at the base to 1.70 m. (5 ft. 6 15/16 in.) at the top, to fill this in with concrete and to repeat the operations 2 m. (6 ft. 6 3/4 in.) higher up after shoreing up the heading by struts bearing against the newly erected concrete wall.

The whole of the work was completed without accident on the 28 November 1923.

[ 625 .154 (.492) ]

# New types of turntable, Dutch State Railways, [1]

By Th. W. MUNDT, Engineer.

Figs. 1 to 4, pp. 434 to 436.

(*De Ingenieur.*)

Up to about 10 years ago, the Dutch State Railways used exclusively turntables with a centre bearing, that is to say, composed of two girders carried on their centre about which they turned. To support the girders when engines were running on and off the turntables, two carrying wheels were provided at each end.

The increasing wheel base of new locomotives called for turntables of larger diameter which, with the corresponding increase in weight of the locomotives, necessitated very deep girders. Centre bearing turntables were constructed up to 20 m. (65 ft. 7 in.) in diameter.

Turntables of this type have the following serious drawbacks :

1. The rails on the turntable have to be set at a higher level than the rails leading on to it, causing shocks when engines pass on and off;

2. Engines have to be accurately balanced on the turntable, which is difficult and wastes time;

3. If the girders are not raised high enough, the table is difficult to turn.

4. The depth of the main girders involves a deep pit (unless a complicated structure with cross girders and stringers is used).

The foundations are expensive, and the girders sometimes not much above water level, which can give trouble in frost.

These disadvantages led to the adoption of the articulated type of turntable.

In turntables of this type, the girders are divided at the middle and connected by a pin-joint arrangement which ensures the wheels at the outer end supporting part of the load. The rails need not be placed any higher on the turntable than the rails leading to it. Engines running on or off the turntable do not cause any appreciable shock, so that the supporting wheels can be provided with ball bearings.

The central bearing consists of a lower bearing with a ball race carrying the upper moving portion fastened to the main girders.

The centre is guided either by a plate fixed to the upper part or a circular journal solid with the moving portion, a loose fit in the lower portion.

With the pin-joint at the centre the girders can be designed as beams supported at two points approximately half the diameter of the turntable apart.

If then :

$l$ = the diameter of the turntable in metres,

$p$ = the weight of the turntable per metre run,

[1] Conference held, 22 April 1926, at the Royal Institution of Engineers, railway construction and operating section.

$q$ = the live load per metre run, the maximum bending moment which is $1/8\ (p + q)\ l^2$ for central bearing turntables will be $1/32\ (p + q)\ l^2$ for articulated turntables.

The depth of the girders may therefore be much less and the pit much more shallow and therefore may not come down to the water-bearing level of the soil.

Another advantage is that the turntable may be turned as soon as all the wheels of the engine are on the turntable. The engine need not be balanced which saves time. Further, articulated turntables may be of smaller diameter than would otherwise be required for engines with their centre of gravity some way from the centre of the wheel base.

In spite of the use of ball races on the articulated turntables on the Dutch railways, the frictional resistance during rotation is greater than in the case of centre bearing turntables carrying a correctly balanced locomotive. Therefore, whenever electric power is available, a motor is used for turning the table. As a standby or in cases where no electric power is available, the table may be turned manually by means of two hand wheels. The table can also be turned by means of levers as used with centre bearing turntables.

Whether worked by hand or by electricity, the power is applied to one of the carrying wheels.

The Dutch State Railways use two types of articulated turntables made by Joseph Vögele of Mannheim and by the « Eschweiler Bergwerks-Verein » of Eschweiler Aue.

The first type is articulated at the centre; the cross bracing is omitted at this point and there is no lateral bracing. The joint transmits the horizontal forces acting on one half to the other half and takes up the inertia forces of the part which is not driven during rotation.

In the second type, the main girders of one half of the bridge are carried on the central bearing beyond which they are slightly extended. These projections support the main girders of the other half. The cross bracing is omitted at this point. Horizontal forces are transmitted from one half to the other by means of a laterally rigid attachment. This attachment is, however, free in a vertical direction so as not to interfere with the articulation of the main girders.

Both types have, generally speaking, proved satisfactory. From a constructional point of view, a few remarks may be made. Certain parts for example are not sufficiently accessible, which prevents their proper upkeep.

The cost of articulated turntables having become high during latter years as a result of the industrial crisis in Germany, endeavours have been made to find a design which, while possessing the same advantages, will not infringe existing patents.

Finally, the type selected was that in which the main girders are carried beyond the centre bearing, on which they rest as the result of the small deflection due to the dead weight (fig. 1).

The central bearing, placed at a lower level than the end bearings (supporting wheels) should be sufficiently low for the unloaded end of the table not to lift when the load is applied at the other end, and at the same time without causing high stresses. In other words, a locomotive turntable must be designed as an ordinary turntable under load, which remains loaded during its rotation.

The main girders may be calculated as beams supported at three points. The centre bearing being lower than the end supports, the girders may be of lighter section than those of articulated turntables of the same span and of the same maximum working stress.

Figures 2*a* and 2*b* allow a comparison to be made of the depth of the girders at the centre for a central bearing turntable 18 m. (59 feet) in diameter and the

Fig. 1. — 20 m. (65 ft. 7 in.) turntable,
Dutch State type, at the Arnhemsche Broek goods yard.

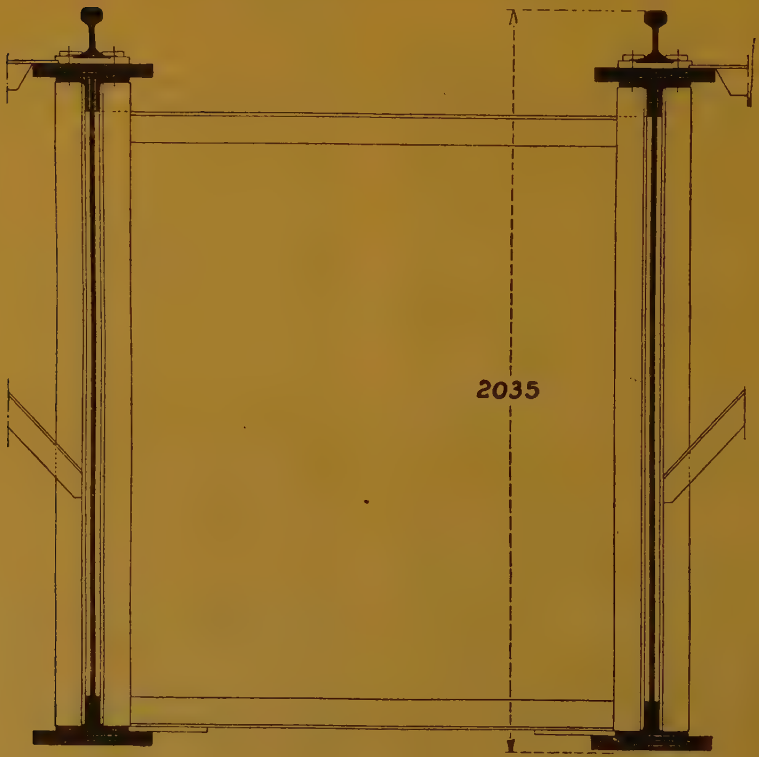



Fig. 2a. — Cross section of 18 m. (59 foot) turntable.

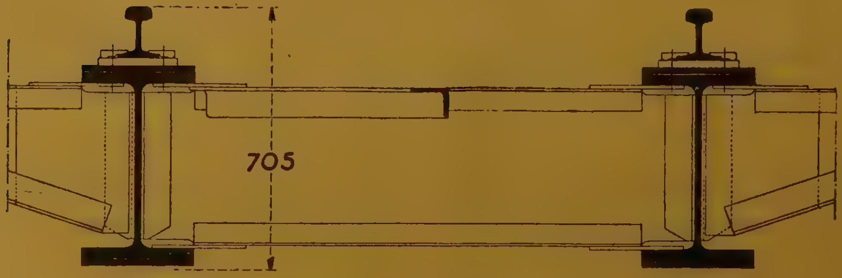



Fig. 2b. — Cross section of 20 m. (65 ft 7 in.) turntable, Dutch State type.

new type of table 20 m. (65 ft. 7 in.) in diameter.

Moreover, the new type has the following advantages over the articulated turntables, especially when dealing with turntables of very large diameter.

1. The cost of construction is less because there is no more or less complicated joint;

2. The rigidity in a horizontal plane is greater and the correct locking of the part indirectly driven by the turning gear is not affected by the amount of play in the horizontal joint. Both ends are locked at the same time by one operation;

3. When the height of the centre bearing is correctly adjusted relatively to the outside bearings, and the main girders are strengthened, the pressure on the bearing wheels during rotation can be made less than with the articulated type. This ensures easy rotation.

In practice, a turntable should also fulfil the following conditions :

1. In the case of only one side being heavily loaded, there should be no undue tendency for the other side to lift;

2. Any sinking of the circular track relatively to the central support should not cause the table to lift. The level of the central bearing should therefore be easily adjustable;

3. When the load is on the side opposite to the driving mechanism, the wheel driven should still carry sufficient adhesive weight;

4. Assuming that the circular track sinks and causes one end to lift, the stresses in the girders at their centre point should still be within the permissible limits;

5. The main girders should be free to come away from the centre bearing so as to avoid additional stress in the main girders should the centre bearing sink relatively to the circular track (fig. 3).

Fig. 3. — 20 m. (65 ft. 7 in.) turntable, Dutch State type, at the Arnhemsche Broek goods yard. Turntable rotating with load all on undriven side.

Four turntables of the Dutch State type, 20 m. (65 ft. 7 in.) in diameter, are in service at the Arnhemsche Broek goods yard, at Sittard, Elst and Lage Zwaluwe. The fifth turntable is ready in the works for the new engine shed at Amsterdam, C. S.

The table consists of a centre portion 17.80 m. (58 ft. 4 in.) long formed of Differdange No. 50 rolled joists covered by a plate 18 mm. (11/16 inch) thick and stayed by means of transverse and horizontal bracing. At each end are a pair of brackets between which are the bearing wheels.

The cross stretchers consist of double D. N. P. No. 50 rolled joists connected to the main girders by means of angle brackets and plates.

The bearing wheels have conical rolling surfaces and turn about fixed axes arranged radially. The pivot pins are carried in bearings secured by four bolts. Each wheel is carried on two ball bearings. The table is turned by applying power to one wheel, this wheel carrying a toothed wheel with which engages the pinion of the driving motor (fig. 4).

The turntable is driven by means of a 10 H. P. motor running at 1 000 revolutions per minute. The motor and controller are cased in.

The motor and all the driving mechanism are placed under the turntable in such a way as to be completely protected against damage should an engine coming on to the turntable be derailed.


Fig. 4. — 20 m. (65 ft. 7 in.) turntable, Dutch State type, at Elst.

1. Suspension bolts.
2. Supporting wheels driven by rotating mechanism.
3. Gear wheel.
4. Motor.
5. Worm wheel casing.
6. Hand operating crank.
7. Handle for engaging or disengaging motor or hand gear.
8. Locking lever.
9. Hand wheel for fine adjustment of position of turntable.
10. Lever for engaging or disengaging this hand wheel.
11. Coupling carrying brake.
12. Brake pedal.

A derailment can occur and often has occurred where several lines lead to a turntable, if the turntable is not correctly placed for the right line.

Three of the turntables have spur wheel drive, the other two having a worm drive.

In addition to the electric drive, the turntable may be operated by hand wheels through chains, or by three levers fixed to the turntable.

To avoid having to overcome unnecessary friction in the latter case, the motor and manually operated gear can both be thrown out of gear.

As has already been said, both sides may be locked by one operation. The bolts cannot be engaged when the table is still turning. The bolts are accurately made to the dimensions of the holes in which they fit, and to engage these requires a certain amount of force. In spite of the provision of a brake on the shaft of the motor, it is not always

possible, when working electrically, to stop the table at exactly the right position for the bolts to engage. For this reason there is a wheel operated by hand by which the turntable can be rotated a short distance on either side of the position in which it stops.

The main girders are fixed at their centre to a strong cross stretcher which rests at two points on the upper part of the centre bearing.

The central bearing consists of a circular plate bolted down to the foundation. On this plate is a ball bearing 800 mm. (2 ft. 7 1/2 in.) in diameter having hard steel balls. The upper portion rotates on the ball bearing. A centre pin carrying no load forms the centre guide. This pin is fixed to the lower portion and is provided with a ball bearing, 180 mm. (7 1/16 in.) in diameter, sufficiently strong to resist the braking effort of a locomotive.

The centre pin is hollow and through it passes the cable supplying the current, which is fixed to a sleeve carried by the pivot. This sleeve also carries the slip rings.

The brushes are fixed to the upper portion. From these brushes, current is taken to the controls by an armoured cable.

The rails on the turntable are carried directly on the main girders on sole plates. Removeable flooring of perforated sheet metal and hand rails on either side are provided.

The rail for the circular track is of the standard No. 46 type, and is carried by special cast iron chairs fastened down to the foundation. The rails are fixed in the chairs by means of keys and T headed bolts, as generally used with the standard No. 46 rail.

The conical shape of the supporting wheels necessitates the rails being canted. The chairs are canted on the foundations to give this inclination. The circular track of the centre bearing is earthed.

The following precautions are taken against any mishaps arising from improper usage.

1. The turntable cannot be locked when the motor is in motion and conversely the motor cannot be started when the locking bolts are engaged;

2. The gearing cannot be engaged or disengaged while the motor is running;

3. The gearing for adjusting the position of the turntable to ensure locking cannot be brought into action while the motor is running, and the motor cannot be started while this gear is in use.

The last two turntables constructed are provided with an arrangement which prevents the brake being brought into action while the motor is working.

The safety devices are placed on the shaft of the controller, to prevent its being improperly used.

Nothing has been spared in the construction as regards quality of workmanship or material, every step being taken to render the turntable satisfactory as regards use and maintenance. Notwithstanding this, we have effected an economy of 40 % on the construction, erection and painting compared with the most recent prices of German articulated turntables. The turntable was, moreover, constructed in our own country. The economy would have been still greater had the specification been no more rigid than for the German articulated turntables.

The first three types of the Dutch State turntables were constructed by Messrs. « Utrechtsche Stroomgrofsmederij P. H. Hörmann » of Utrecht, the last two by the « Nederlandsche Machinefabriek » of Windschoten.

This last type in practice fulfils all the conditions laid down.

In addition to the 20 m. (65 ft. 7 in.)

turntables constructed, we are shortly commencing the construction of a 24 m. (78 ft. 9 in.) turntable of the Dutch State type for Zevenaar. This turntable will be used for turning large German locomotives, and is designed to carrry engines weighing 275 t.

If only four carrying wheels were used, the pressure on the wheels would be extremely great. This would lead, not only to considerable expense in laying the rails and constructing the foundations of the circular track, but also to rapid wear of the rail and wheels. This turntable has therefore been designed with eight wheels, four at each end.

The wheels are in pairs carried in a cast steel frame. Each end of the turntable is carried on these frames in such a manner that by deflection of the supports the load may be equally distributed on each of the wheels.

The drive from the motor will be transmitted to two wheels in one frame; the hand gear will only act on one of the wheels.

The drive from the motor is applied to two wheels because the central support would need to be placed too high for easy operation by hand.

Moreover, if only the end not driven by the motor were loaded, the adhesive weight on one wheel would be insufficient to rotate the turntable.

However, in the case of the very heavy loads contemplated the maximum pressure on the wheels will amount to 30 t., therefore it was considered necessary to provide a differential mechanism. This will better ensure easy running in the case of unequal wear of the tyres and avoid damage to the tooth wheels or pinions.

The drive is effected by a combination of gear wheels and worm drive. The construction is identical in principle with that of the 20 m. (65 ft. 7 in.) diameter turntables.

# MISCELLANEOUS INFORMATION

[ 621 .138.2 (.42) ]

## 1. — Mechanically-operated locomotive coaling plant, London Midland & Scottish Railway.

Figs. 1 to 3, pp. 440 to 443.

(*The Railway Gazette.*)

In connection with the reconstruction programme at the Polmadie running sheds, Glasgow, of the London Midland & Scottish Railway, a locomotive coaling plant of new and interesting type has been recently installed and put into commission. In the design of this plant particular attention was paid to the following points :

1. The possibility of storing sufficient coal to make it unnecessary to employ labour at night or on Sundays for the purpose of discharging coal wagons;
2. Handling the coal as it comes from the collieries without the necessity of passing it through a crusher;
3. Rapidity of operation with a minimum of power and labour;
4. Limitation of the ground area to be occupied by the plant and the cost of the foundations.

The general design will be seen from the photographic views and drawings reproduced on following pages.

The plant consists of two steel bunkers situated on either side of a central hoist shaft, which is surmounted by the house carrying the wagon-elevating machinery. Each bunker is capable of taking 150 tons of coal, and it is intended under normal working conditions to carry grade I coal in the one bunker and grade II coal in the other. The bunkers are each provided with two discharging gates and chutes, and it is, accordingly, possible for as many as four locomotives to draw up alongside for coal at the same time, *i. e.*, two on each side. The weight of coal supplied to the locomotives is measured by the boot or lower part of the chutes being dimensioned to contain 10 cwt. at a time. The flow of coal into the boot is regulated by means of a gate, and it is in turn released from the boot by means of a drop-extension chute, which directs the coal on to the tender or bunker of the locomotive. The bunker gates, as well as the drop chutes, are operated by means of levers working in quadrants, weights being provided to counterbalance the weight of the moving parts. A simple interlocking device makes it impossible for the drop chute to be lowered unless the gate is closed.

### Hoisting gear.

The hoisting gear is designed to handle standard end door trucks of a wheelbase up to 9 feet. The maximum load to be hoisted is 20 tons at a speed of 25 feet per minute. Two Metropolitan-Vickers totally enclosed series-wound, reversing-type motors drive the hoisting mechanism, the current supply being 500 volts d. c.

Control of the hoist motors is effected from a cabin situated at one side of the central shaft and at a height of 49 feet above ground level. Two controllers of Allen, West & Co.'s enclosed drum type make, operated by means of a single crank lever, serve to control the hoisting and lowering motions. These controllers are provided with a sufficient number of contacts in either direction to ensure steady starting and even acceleration of the motors.

The motor circuits are protected by means of double-pole circuit breakers fitted with pow-

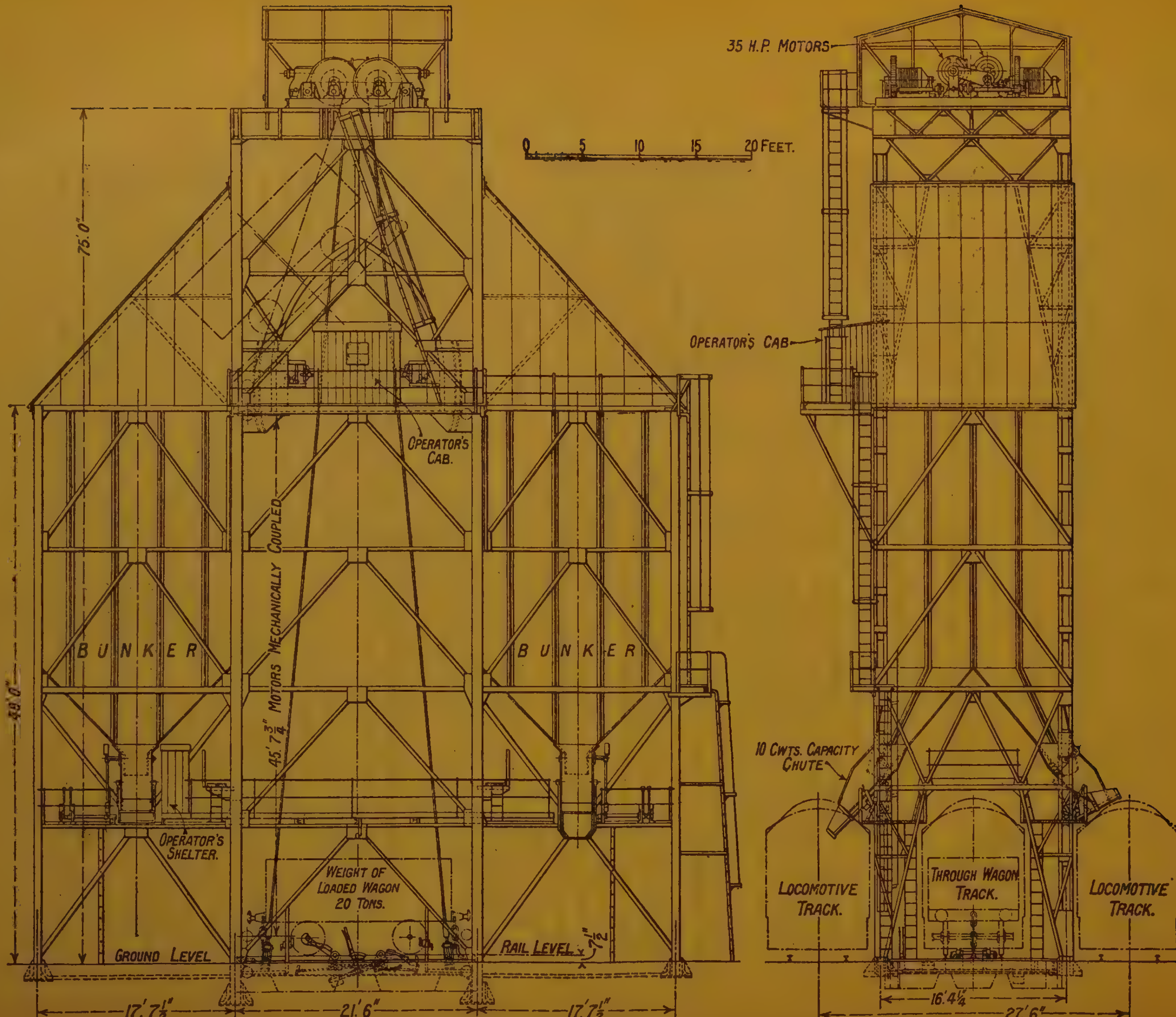



Fig. 1. — Side and end elevations of mechanical coaling plant at Polmadie sheds, Glasgow, London Midland & Scottish Railway.

erful magnetic blouw-outs and overload releases. Self re-setting limit switches are provided to prevent overwinding of the cradle at the top of its travel, and also to cut off the current when the cradle is brought to rest at the foot of the hoist.

The cradle is strongly built up of structural steel joists and sections covered with chequer plating. When at the bottom of the hoist shaft the cradle rests on concrete piers, thus relieving the hoisting ropes of any strain when wagons are being run into position for hoisting, or empties are being run off. The heaviest main-line engines can also pass across the cradle should it at any time be necessary for them to move over the central track from one end of the plant to the other.

Locking of the trucks on the cradle is effected by means of two pairs of locking bars, only one pair of which requires to be employed at a time. These bars are raised and lowered by means of hand levers operating in quadrants, with counterweights situated under the cradle to balance the weight of the bars. The latter are provided with hooks which engage with the wagon axles. Incoming wagons are run against the hooks situated at the end of the cradle in the direction in which the wagon is to be tipped. As the axle passes into the hooks, keep plates situated on the locking bars immediately behind the hooks rise automatically, effectually locking the axle, and preventing any return movement of the wagon. Disengagement of the wagon axle after discharging is made by moving the hand lever over and dropping the locking bars to the platform level.

### Special features of control.

Movement of the locking-bar levers also automatically actuates a switch which determines the direction in which the wagon is to be tipped. That is to say, the wagon can only be tipped towards the end at which it is locked, and where the door is situated. As soon as a wagon is locked and ready to ascend, the attendant communicates with the operator in the cabin by means of an electric bell. The hoisting motors are then set in motion and the cradle is raised in a horizontal position to a height of about 51 feet. At this point the hoist motor on the side towards which the wagon is to be tipped is automatically cut out, and the cradle continues to rise at the other end only, entering curved guides at the same time, which regulate its position and travel. In order that the catches holding the wagon door may be freed, the operator moves his controller over to the off position, as soon as the cradle has reached a convenient height. Thereafter the power is again switched on, and the wagon tilted to a sufficient angle to allow the coal to fall into the bunker. The maximum angle of tilt provided for is 45°, at which point the previously mentioned overwinding switch comes into action, cutting off the current. On the lowering side only one motor is set in motion until the cradle resumes the horizontal position, when the second motor comes automatically into action. It will thus be seen that the operator has merely to start and stop the hoisting or lowering motion of the cradle by means of his controller handle, the tilting effect being actuated entirely automatically. In order to ensure that the speed of the two motors when operating together is equal, a special device is employed by means of which the two motors are mechanically interlocked as soon as the cradle has been raised to a height of 3 ft. 4 in. from its base, and until the point of tipping is approached. A similar effect is obtained when lowering, and the cradle accordingly always maintains a more or less horizontal position.

The cradle is suspended from the hoisting mechanism by means of four wire ropes attached by shackles free to swing on their axles. Powerful screws allow for adjustment of the ropes as required. The rope drums, four in number and arranged in pairs, are driven by the 35 H. P. motors through worm reduction gears operating on to horizontal reduction shafts, which carry pinions at each end gearing with spur wheels mounted on the drums.

Powerful solenoid-operated brakes are fitted to each motor, and are so connected that the current passes simultaneously through the solenoid and motor to release the brake. Thus

Fig. 2. — General view of coaling plant at Polmadie. Glasgow. London Midland & Scottish Railway.

the brakes would come into action immediately should the current be cut off or fail from any cause.

In addition to the solenoid brakes, the hoisting mechanism is arranged for dynamic braking control on the potentiometer system in the

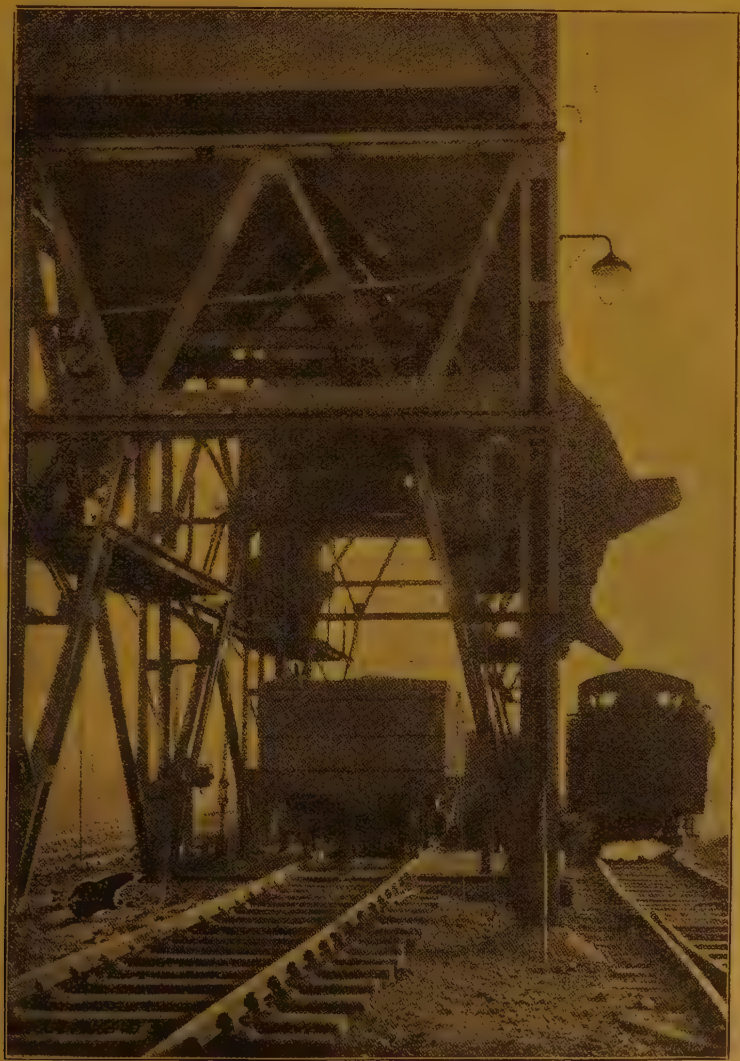


Fig. 3. — Rail level view of plant in service.

lowering direction, giving complete control of the speed of lowering with any load and on any notch of the controllers, the method being that, on the lowering side, the armature is put in parallel with the series field and a separate resistance. The motors accordingly regulate

themselves automatically and the load is either driven down or, if sufficiently heavy to drive the gear, the motor becomes a generator and the energy of the falling load is dissipated in the resistance. The motors are never cut off from the line and the load is always completely under control. The maximum lowering speed on the last notch with full load would not exceed twice the normal full hoisting speed. There is thus no danger of any damage being caused to the motors by excessive speed even should the controller be handled by an inexperienced operator.

With regard to the speed of working of the coaling plant, the time required to raise a loaded wagon, discharge, and return to the bottom averages 4 1/2 minutes. With an additional 1 1/2 minutes for removing the empty wagon and placing another one in position on the cradle, 10 wagons can be handled per hour. Assuming that each wagon carries only 10 tons of coal, this gives a hoisting capacity of 100 tons per hour. Under normal conditions the hoisting equipment can handle sufficient coal during the day shift to supply a very large number of locomotives coming in for coal at any time spread over the 24 hours.

The mechanical coaling plant was constructed and erected under contract, to the design and requirements of the company's Chief Mecanical Engineer, by the Wellman Smith Owen Engineering Corporation Limited, Victoria Street, London, S. W. 1.

---

[ 621 .99 (.73) ]

## 2. — Lock nut made with new thread form.

Figs. 4 and 5, pp. 444 and 445.

(*Railway Age.*)

The Graham Bolt & Nut Company, Pittsburgh, Pa., has recently announced the production of a lock nut with a self locking form of thread, to be known as the Selflock. This form of thread is a mechanical development of screw threads to produce a locking element in nuts, which can be used in conjunction with bolts having United States Standard threads. In developing this thread as many of the characteristics of the U. S. S. thread were maintained as possible, resulting in the following common characteristics: Equal areas, true U. S. S. lead, location of pitch line and U. S. S. flats.

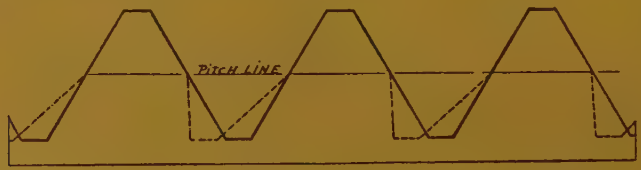



Fig. 4. — A comparison of the U. S. S. thread with the thread form used in Selflock nuts.

One of the illustrations shows clearly a comparison of the Selflock thread with the U. S. S. The pitch line location is the same and it will be noticed that the thread form inside the pitch line is identically the same as U. S. S. while outside the pitch line the thread angles are slightly changed.

In the design of this lock nut there is no distortion of the thread, the lead being true U. S. S. and the helix angle constant. Due to the equal thread areas no material is removed when Selflock nuts are applied on U. S. S. threaded bolts. The lead being true U. S. S. the nut may be applied from either face, the holding power developed being the same in either case.

When Selflock nuts are applied, that part of the thread on the bolt lying outside the pitch line is slightly tipped so as to create a definite frictional lock on every thread engaged by the

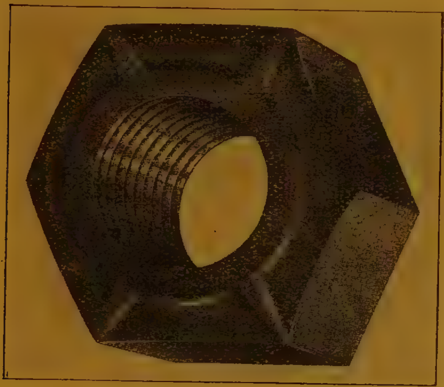


Fig. 5. — Selflock nuts may be identified by this star crown.

nut. Should it be necessary to remove the nut it will require more wrench load to break the contact and start the nut off the bolt than was required to put it on. The Selflock nut is applied in the usual manner — started with the fingers and wrenched to position.

U. S. S. nuts may be applied to bolts having had Selflock nuts on them. Should the U. S. S. nut be reasonable in size, a certain frictional lock will be developed as it will be necessary to bring the thread on the bolt back to suit the U. S. S. conditions in the nut. Selflock nuts may be applied many times to the same bolt thread. Should this be done so many times that sufficient lock is not developed, the nut can be reversed when the locking feature will again function.

Because of the fact that the locking feature of the Selflock nut is cut into the solid metal of the nut, it is impossible for the workman to change its locking value in any way — the nut cannot be normalized.

The manufacturer has been experimenting with this type of lock nuts for some time and actual service tests on locomotives, freight cars, frog and crossing bolts, track bolts, forging machines, cranes, rolling mill equipment and similar installations where severe vibratory conditions prevail are said to have demonstrated satisfactorily the ability of this type of thread to resist the loosening effect of vibration. The star crown shown in one illustration has been adopted as a means of identifying these nuts.

---

[ 625 .216 ]

## 3. — Automatic coupler trials at Trappes. — Tests of the « Willison » coupler,

By P. SUTRA.

Figs. 6 to 19, pp. 445 to 450.

(*Les Chemins de fer et les Tramways.*)

Thirty years ago the question of automatic couplers was a very live one in railway circles in the United States of America, where automatic couplers are now used exclusively. Many new countries, from a railway point of view, such as Japan and British India, have adopted automatic couplers because of the greater safety in working and the saving they give in shunting operations.

Their use on the more or less old European rolling stock, which is also of many types, is under consideration in the chief countries of the old world. As the result of commercial and political pressure during the last two years, their use on the French systems has been considered more closely. The State Railways is investigating the matter, and many tests have been carried out both at Trappes and at La Rochelle with two types : the « Boirault » and the « Henricot ». At the same time England and Germany considered various designs and, amongst others, the American coupler,

the « Willison », put forward by the Knorr Company in Germany and Cammell Laird in England.

In view of the results obtained abroad with the « Willison », later adopted by the « Bombay-Poona » line in British India, the French

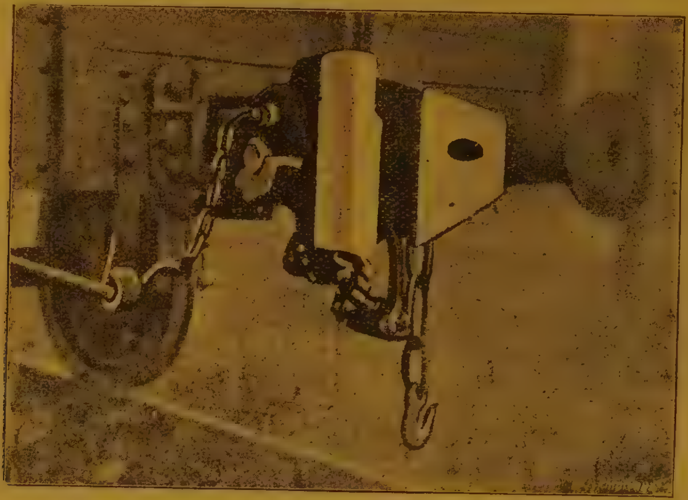


Fig. 6. — Coupler head. — View from cam side.

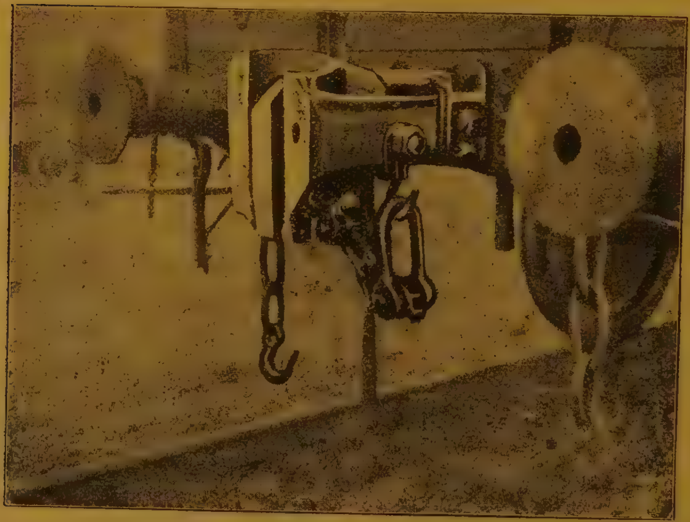


Fig. 7. — Coupler head. — View from side, screw coupling hooked up.

Company « Les Appareils Ferroviaires » has undertaken at its own cost to demonstrate this coupler to the French railways. Its initiative was successful as the State Railways has

agreed to tests being made in competition with the two types which, so far, were the only ones under consideration.

A difficult test programme was drawn up by the Central Office for the design of Rolling Stock. The tests with the « Willison » coupler were completed at Trappes on the 25 January 1926 under very severe weather conditions. Representatives of the railway companies and of the narrow gauge lines, as well as a number of technical experts were present.

* * *

The « Willison » coupler is an automatic coupler with centre buffer suitable for use on all lines of all gauges, and on all stock, both goods and passenger. During the transition period this coupler is the only one that will couple properly with the different types of drawgear and buffering gear in use on the different companies without any previous adjustment.

The coupler has no delicate parts, contains no springs, and its parts require no greasing.

As figures 8 to 11 shew, it is composed of two main parts in cast steel, a jaw with two lugs A and B and a lock C sliding at an angle of 45° in the head of the coupler itself, which drops by its own weight and prevents uncoupling.

Uncoupling, which can only be done deliberately, is effected by pulling a chain D on the side of the vehicle which rotates a cam F about a spindle E carried through the head of the coupler and lifts the lock; the two vehicles then uncouple. If the chain is left loose, the vehicles can then be recoupled. If the chain is held by a stop on the bracket on the frame, the cam holds the lock up and coupling cannot be done. This arrangement is necessary for shunting in marshalling yards. During the transition period, the pin E also carries the screw coupling : this will be dealt with later.

The head of the coupler is hinged at G on the frame to give the necessary play when going round curves : the lateral movement is limited by a guide carried on the headstock of the vehicle with an opening determined by the width of the vehicle, the gauge, and the radius of the curves, so that in all positions the vehicles can be coupled together. For standard gauge lines the play amounts to 127 mm. (5 inches) on each side of the centre line of the draw gear.

Figures 12 and 13 shew the extreme positions on a curve of two couplers with side movement limited by the guide on the head stock, and figures 14 and 15 shew the relative movement of the two coupler heads, to allow them to couple. This movement is due to the shape of the heads and their sliding faces, which are given a slight incline to ensure the heads engaging with one another in all positions.

To allow the maximum vertical movement when running over a badly laid track, or when coupling wagons with different heights of draw gear like, for example, the French and American wagons in use on our lines since the War, the total vertical movement may be 250 mm. (10 inches) which this coupler alone can meet. This difference actually occurred at Trappes on a short line specially built to shew this action on a gradient of 30 mm. (1 in 33) on both sides.

The surfaces taking the blow are vertical and plane. A slight convexity in the vertical plane flattened out at the centre is given on the faces taking the pull and allows the couplers to take up a position slightly out of vertical with a minimum of play between couplers.

* * *

The coupler works as follows : The « Willison » automatic coupler is always in the ready-to-couple position (except in the special case dealt with below); under thrust, the inclined surfaces of each head strike the opposing faces or, under some conditions, as on curves, the lock of the coupler head in question. The heads move until the locks which have been forced back on to their seats, and thereby have allowed each knuckle to enter the opposite jaw, can fall into their first position (figs. 16 and 17), and thereby ensure coupling, uncoupling being impossible except by deliberate action.

To uncouple, the outside lever, placed on the side of the wagon, acts on the cam which, lifting the lock releases the jaws, and thereby

Fig. 8.

Fig. 9.

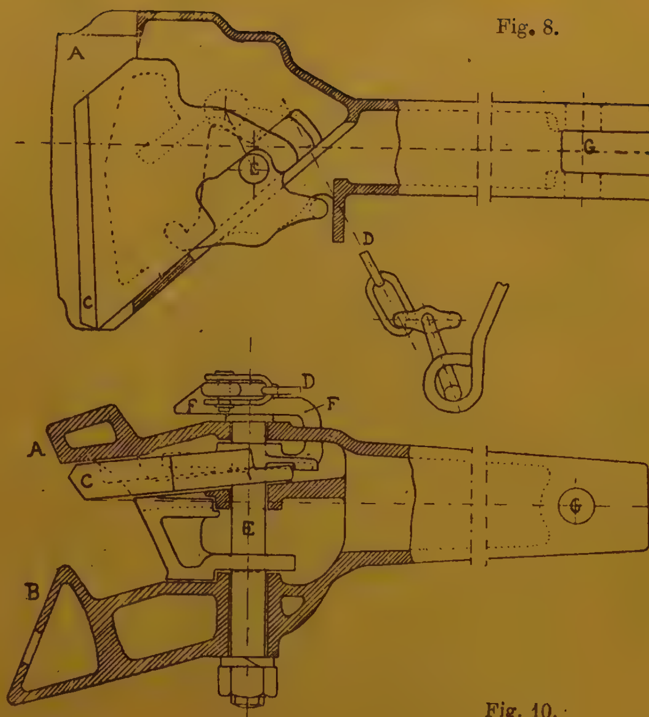



Fig. 10.

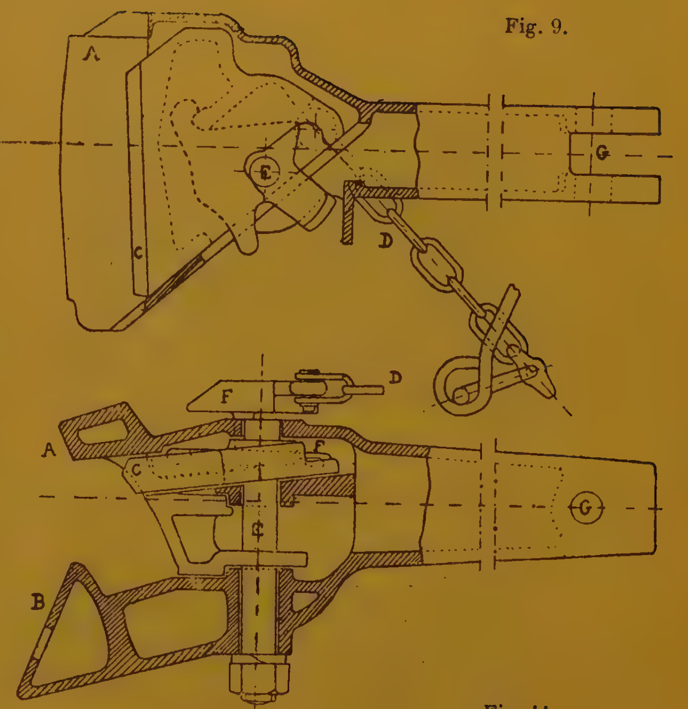



Fig. 11.

Figs. 8 and 10. — Horizontal and plan views : position in which it will not couple.

Figs. 9 and 11. — Horizontal and plan views : position for coupling.

Figs. 8 to 11. — Details of the coupler head.

Fig. 12.

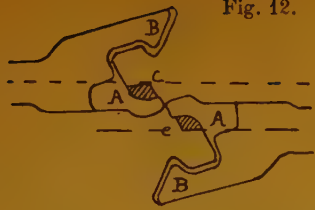
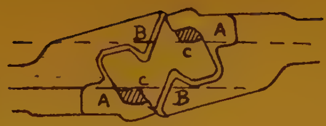



Fig. 13.



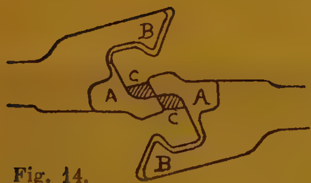




Fig. 14.

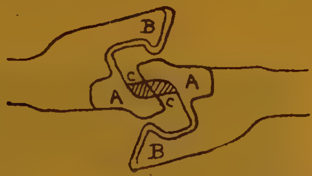



Fig. 15.

Figs. 12 and 13. — Relative outside positions on a curve.
Figs. 14 and 15. — Movement of heads to allow coupling.

Figs. 12 to 15. — Stages of coupling


Fig. 16.

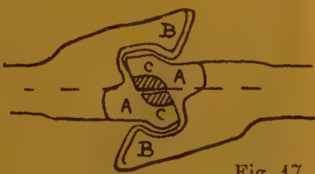



Fig. 17.

Fig. 16. — Two heads entering one another with wedges being pushed back.
Fig. 17. — Coupling completed by wedges dropping.

Figs. 16 and 17. — Stages of coupling.

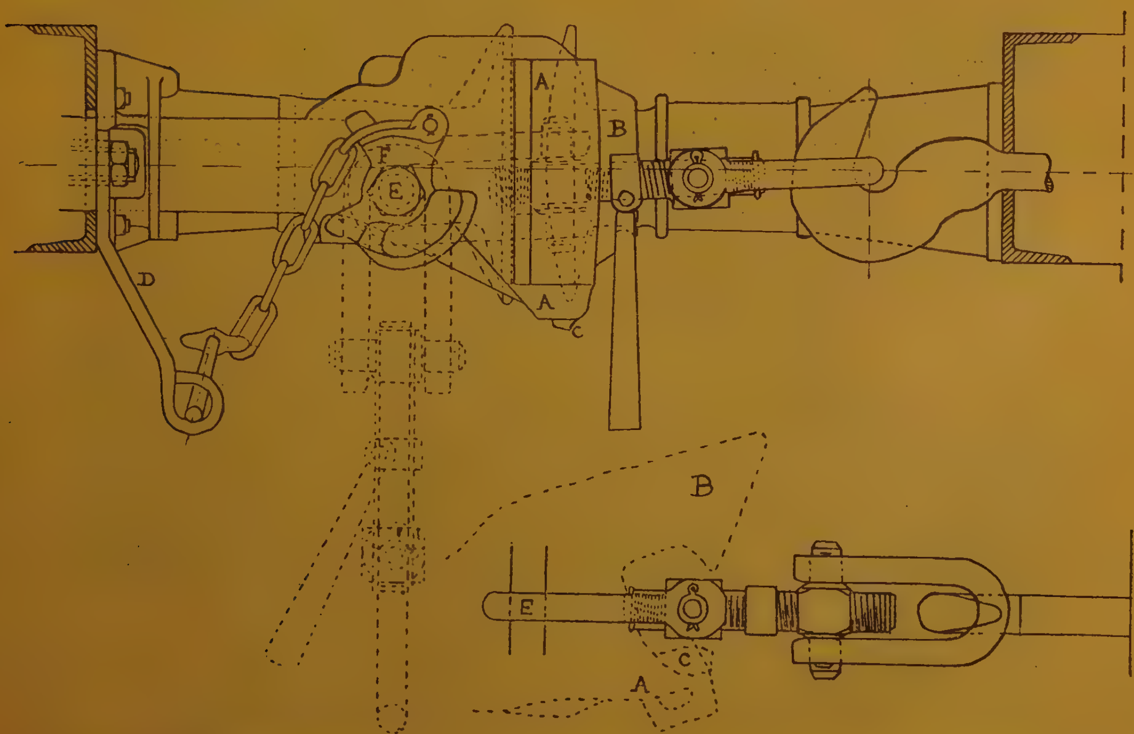



Fig. 18. — Arrangement of coupling by hook and screw coupling during transition period.

uncouples the wagons. The lock falls under its own weight to the normal position for coupling.

Certain shunting operations in marshalling yards make it necessary to shunt wagons without coupling them together. This is done by lifting the lock and holding it up by the lug on the pin when the lever is held in the stop.

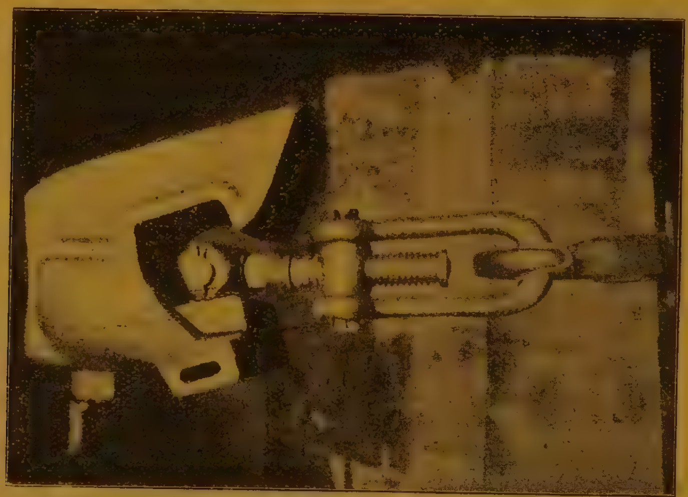


Fig. 19. — View of coupler head with screw coupling in place during transition period.

The « Willison » coupler is shewn by the above description to be the only one in which pulling and buffing stresses are taken by the head itself, that is to say, by a rigid cast steel part, and not by moving parts, such as locks or pins, as in all other couplers. Each automatic coupler therefore forms a single piece; buffer and buffing is transmitted equally on both sides of the centre line of the draw gear, and directly from head to head of the couplers.

Traction is also taken directly by the heads and pins in the same way through ample surfaces at equal distances on each side of the centre line.

* * *

The present conditions and the amount of traffic handled would not allow our Companies to change over quickly. The makers of this coupler have made provision during the transition period for the coupling of a vehicle with automatic couplers to others with the ordinary hook and screw coupling without any special preparation. The « Willison » automatic coupler is fitted provisionally with an ordinary screw coupling fastened to the coupler head in line with the draw gear, as shewn in the drawing (fig. 18).

* * *

As stated at the beginning of this article, this coupler has just been tested during very severe weather and passed the tests, both for safety and for strength. In addition, pull tests were carried out at the Conservatoire des Arts et Métiers. A pull of 70 t. did not distord the couplers in any way. The coupler broke at 140 t., so there is an ample margin of safety. So far as resistance to a blow is concerned, the head broke at the 21st blow from a tup of 800 kgr. (1 760 lb.) falling 3 m. (9.8 feet).

We think and hope that the official figures which will be communicated shortly to the French Commission on Automatic Couplers will place the « Willison » amongst the leading makes to be considered for use by the French railway systems.

[ 656 .253 (.44) ]

## 4. — Signalling of railways by light signals day and night,

By J. NETTER.

Figs. 20 to 26, pp. 451 to 453.

*(Le Génie Civil.)*

The signalling of railways during daylight is generally assured by mechanical signals, such as discs of different colours, or semaphores, which can be set in different positions; whilst at night light signals are used, the colour being altered by moving coloured glasses in front of the lamps. Light signals can be seen at night much farther than mechanical signals can be sighted during daylight, and their control from a distance obviously requires much less effort than is required to operate mechanical signals, such as discs or semaphores. The exclusive use of light signals would be of undoubted advantage if their visibility during daylight could be made at least equal to that of the mechanical signals.

The problem was solved some years ago in America, and more recently in France, by using a source of light of high candle power per square millimetre, such as electric lamps with tungsten filaments, and by placing the light at the focal centre of a stepped lens no thicker at the centre than at the sides. A lens of this kind can be suitably coloured without too great absorption of rays through the centre part as with ordinary lenses.

The absorption will be a minimum if care is taken to determine, from a careful analysis of the source of light, the shade to be given to the lens to obtain the desired colour. In this way blue predominates in the glass which shews a green light. Lights of this kind are visible in daylight even in very clear weather, for over 600 m. (1968 feet), and can be used in place of mechanical signals with advantage wherever current for the electric lamps can be readily obtained locally. Should the local supply fail, the difficulty is easily overcome by the use of an automatic generating set, or a reserve accumulator battery.

The consumption of the lamps used has been reduced to 16 watts, at 8 volts. The lamps supplied by the Compagnie Générale de Signalisation have two compact filaments, arranged in parallel, so that if one fails the other continues to burn. This arrangement appears to be better than that in which one only of the filaments is in circuit, the other being spare, and coming into use automatically if the first fail. The filaments are known to be much stronger when hot, and are more likely to break when current is not flowing.

Figure 20 shews the optical system of two combined lenses used by the Compagnie Générale de Signalisation. The exact centering of the lamp at the focal centre of the optical system is obtained by means of three tenons placed at varying distances on the cap of the lamp which, by exact location of the ring, assure the filaments taking up the correct position. The centering of the filaments in the lamp is looked after during assembly and sealing of the cap, which are done in a special machine. In this way no special precautions are necessary when replacing a lamp.

Fig. 20. — Section of an optical system consisting of two stepped lenses.

The lamp is carried by a metal plate carrying the ring and fastened without any means

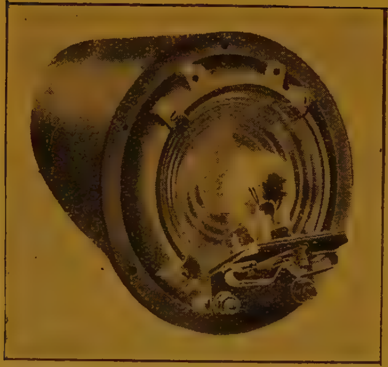


Fig. 21. — Back view of light signal.

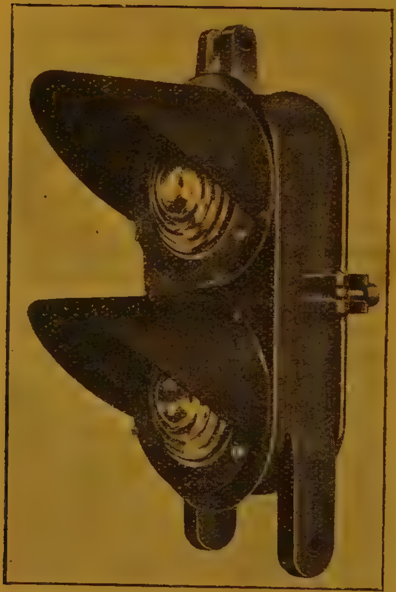


Fig. 22. — Double light signal panel.

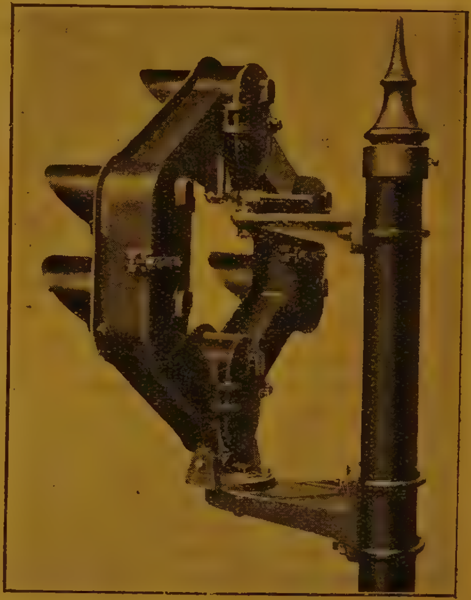


Fig. 23. — Arrangement of six light signals on adjustable bracket.

of adjustment to the ring carrying the lenses. As the setting is done once for all at the works, the rays of light are exactly at right angles to the ring carrying the lenses. This ring at the top has a hood to shield the lens from outside reflections.

The single signal unit made up of a lamp and its lens (fig. 21) is fixed in one of the openings of a cast iron box with a back like a shutter with ventilating openings in it (fig. 22). A shade in steel metal of suitable size is fastened to the front of the casting. When the signal has to shew two lights, as with most signals in France, the panel is formed by a

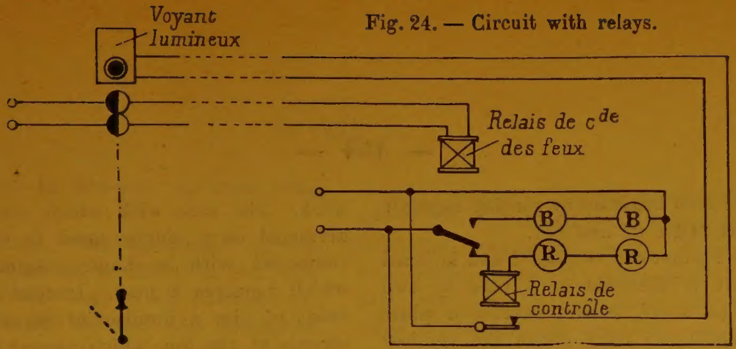



Fig. 24. — Circuit with relays.

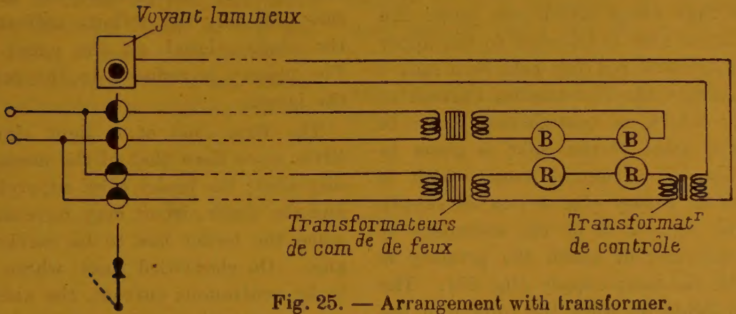



Fig. 25. — Arrangement with transformer.

**Figs. 24 and 25. — Diagram of operation and control of signal lights.**

B = White; — R = Red.

*Explanation of French terms* : Relais de c$^{de}$ des feux = Relay operating the signal lights. — Relais de contrôle = Control relay. — Transformateurs de com$^{de}$ de feux = Transformer operating signal lights. — Voyant lumineux = Light signal.

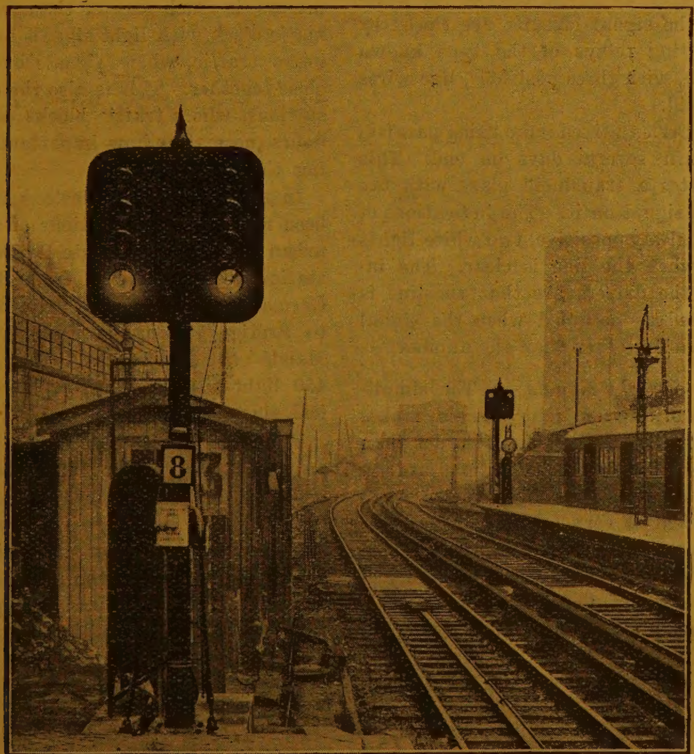



Fig. 26. — Light signals, French State Railways.

frame in cast iron made up by joining together several boxes (figs. 22 and 23).

So that the panel can be placed and inclined as desired, it is fastened to a tube by two brackets. The lower bracket carries a plate with lugs which can turn about the vertical axis (fig. 23) moving with it the horizontal pin through the lugs about which the panel can turn. The latter also is fastened to the upper bracket by a screwed rod and held by a nut.

The lamps take the low tension current of the company which is transformed down to 16 volts. The state of the light is given in the signal box by a control lamp, which is lighted either by a reley (fig. 24) in the supply circuit to the signals, or by the secondary of a small transformer of which the primary is in series with the lamp supply (fig. 25). The control lamp lights up when the voltage of the supply to the signals drops by half, which corresponds to one of the filaments of the signal lamp breaking. The signalman has to replace the defective lamp, although it is not necessary to do it immediately.

When the signals are controlled automatically by the trains, as in the case of the automatic block, the signal circuits are closed by relays. By using relays of the type known as the « Disc », with three positions, line wires need not be laid.

The signal carries a fixed lamp using paraffin which can burn several days on end. This lamp illuminates a translucid glass with the number of the signal on it. The indications of the signal are always positive, two white lights being shewn when the line is clear. The instructions to the drivers are that the line is to be considered as occupied when the signal shews no indication other than the number.

*Advantages of light signals.* — The simplicity of the working of light signals makes them much less likely to get out of order than mechanical signals. Their indications are always definite as the white light shewn when the track is clear is very easily distinguished from any other light the driver may see. In fog their visibility is three times that of mechanical signals, and in clear weather it is at least as good. They are much easier to work. The ease with which several can be arranged on a single panel is very valuable compared with mechanical signals, each of which requires a post. Instead of seeing in front of him a number of signal posts with signals at the top, which appear to be giving contradictory indications, the driver has before him only the definite indication given by the single signal on the panel illuminated. The upkeep is reduced to the replacement of the lamps.

The first cost of a light signal panel is little more than that of the mechanical signal displaced; the feeder cable supplying the lamps and the track circuit may increase this outlay when the feeder has to be carried some distance. On electrified lines, where the traction is by continuous current, the automatic block system necessitates the use of alternating current which involves a special main to feed the track circuit and the lamps. On steam lines alternating current is safer than continuous because of the danger of parasitic currents. The cost of installation of the automatic block with mechanical signals operated by electric motors, is much greater than that of the automatic block with light signals on all lines with heavy traffic, where signals have to be fairly close together. This is also the case near main stations, where traffic blocks occur at certain hours, no matter how important the lines ending therein may be.

In France such installations have so far only been made on some sections of the Paris Suburban lines : on the State Railways (Invalides station to Meudon, and from Bécon to Saint-Germain); on the Orléans System (from Paris to Brétigny); on the Eastern (from Saint-Mandé to Vincennes). The total exceeds 450 light signals which replaced 950 mechanical signals.

When the Minister of Public Works visited, on the 13 September 1926, the installations made by the State Railways, such as shewn in figure 26, the daily press pointed out the advantages obtained from the point of view of safety by the use of the automatic block system with light signals, but expressed a fear that the cost would militate against their ex-

tended use. This is, however, far from being correct.

As an example, the equipment of the 5.200 km. (3 1/4 miles) section from the Invalides station to Issy cost in all 1 200 000 fr., that, is about 230 000 fr. per kilometre. Against this, the annual cost of upkeep of the new block system, including current, amounted to 80 000 fr. On the other hand, on this section an annual saving of more than 400 000 fr. was effected by doing away with 35 men exclusively employed beforehand in working the mechanical signals. The net annual saving was 320 000 fr. which was sufficient to pay off the total first cost in four years. This amortization would be spread over a longer period in most cases, as the signalmen also carry out other duties as, for instance, guarding level crossings. The advantage of assuring the protection of level crossings by automatic signals, such as the « Wig-Wag » (1) is evident.

(1) See the *Génie Civil*, 7 November 1925, Vol. LXXXVII, Number 19, page 398, for a description of this system.